ANDERSON

The Effect of Position & Spacing of Reinforcing Bars in Reinforced Concrete Beams

Architectural Engineering

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THE EFFECT OF POSITION AND SPACING OF REINFORCING BARS IN REINFORCED CONCRETE BEAMS

BY

RUSSELL ADAMS McCURDY ANDERSON

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

ARCHITECTURAL ENGINEERING

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COLLEGE OF ENGINEERING

June 1, 1911.

This is to certify that the thesis prepared in the Department of Theoretical and Applied Mechanics by RUSSELL ADAMS MCCURDY ANDERSON entitled The Effect of Position and Spacing of Reinforcing Bars in Reinforced Concrete Beams is approved by me as fulfilling this part of the requirements for the degree of Bachelor of Science in Architectural Engineering.

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I. INTRODUCTION.

1. PRELIMINARY.

It was with the idea of obtaining better knowledge about the proper position and spacing of the bars in reinforced concrete beams that this thesis was undertaken. In practice the reinforcement is placed in different ways and positions and the investigation entered into was to ascertain the effect of varying the placing and spacing of the bars, on the horizontal shear and the bond.

The following division of the subject matter has been made; I. INTRODUCTION; II MATERIALS, TEST PIECES, and METHODS of TESTING; III EXPERIMENTAL DATA and DISCUSSION.

The first series of fourteen beams was a study of the spacing of the bars and the horizontal shear. The other eight beams was a study of the effect on bond strength of the bending up of the bars at various intervals and consequently of how the stresses were distributed among the bars.

Several of the test pieces of the same kind gave results that differed quite extensively. This was probably due to the difference in the quality of the concrete and the way in which it had been put in the forms. The beams of the first series were each tested in the same machine and under the same conditions as nearly possible. The first four of the second series of beams were tested in the 600 000 lb. Rhiele machine and the last four beams were tested in the 200 000 lb. Olsen machine. These machines are in the Laboratory of Applied Mechanics.

2. SCOPE of THESIS.

The investigation included the testing of twenty-two beams in two series, the first consisting of fourteen beams, the second of eight beams. The subjects investigated are respectively,

Series I. Horizontal Shear Between Bars.

Series II. The Effect of Bending Up Reinforcing Bars near the Load Points.

3. ACKNOWLEDGEMENT.

Much help was given to the writer by Mr. W. A. Slater both in testing the test pieces and in the working up of this thesis. Messrs. Lord and Abrams also gave their assistance in running the 300 000 lb. Rhiele machine when the first four beams of the second series were tested.

II MATERIALS, TEST PIECES and METHODS of TESTING.

4. MATERIALS.

The materials used were such as are used in ordinary concrete construction in the Middle West. The sand and stone were obtained in the open market. The cement was obtained from the Universal Portland Cement Co. and the Lehigh Portland Cement Co. The steel was furnished by the Illinois Steel Co. The sand was of good quality from near the Wabash river at Attica, Indiana. It was fairly clean, sharp and well graded and weighed about 100 lbs. per cu. ft. The fineness of the sand is given in Table I, the average of eight tests.

The stone was a good quality of rather hard limestone from Kankakee, Ill. ordered screened through a 1 in. and over a one-quarter in. screen. Its fineness is given in Table II

TABLE I.

Mechanical Analysis of Sand.

Sieve No.	Sample No. 1	2	3	4	5	6	7	8
	Per cent Passing.							
3	100.0	100.0	100.0	99.7	100.0	100.0	99.0	97.8
4	99.1	98.9	99.1	99.0	99.2	99.4		
5	95.3	95.1	95.7	94.1	98.5	98.1	98.0	98.8
10	65.4	63.7	65.9	63.7	84.5	84.1	82.6	84.8
12	58.1	55.7	57.7	55.6	78.4	78.5	76.8	78.7
16	51.9	48.7	51.3	49.3	72.0	72.3	70.1	72.1
18	42.5	39.9	41.9	40.5	60.2	59.7	57.1	59.9
30	26.6	24.6	26.3	25.3	33.3	33.5	39.3	34.0
40	14.5	13.0	14.7	13.9	19.7	19.9	19.3	20.1
50	5.4	4.6	5.9	5.4	8.2	8.7	8.7	9.0
74	2.1	1.7	2.3	2.3	3.7	3.9	4.0	4.3
150	.8	.5	.8	.8	.7	1.0	1.3	1.1

TABLE II.

Mechanical Analysis of Stone.

1	100.0	100.0	100.0	100.0	100.0	100.0		
3/4	95.7	97.0	95.2	94.1	95.3	94.3	98.4	91.1
1/2	63.3	61.0	56.2	60.9	58.5	57.6	60.7	41.2
3/8	38.7	40.4	33.3	36.7	30.3	30.9	35.0	16.7
No. 3	17.7	23.4	17.6	19.9	13.7	15.3	16.5	7.0
5	2.9	5.0	6.0	4.7	2.3	1.6	2.8	2.2
10	2.1	3.1	4.3	3.2	1.8	1.1	1.8	2.0

Universal and Lehigh portland cement were used the following table giving the results of fineness and tensile strength.

TABLE III.

	7 day		28 day	
Lehigh	Neat	1:3	Neat	1:3
	719	248	805	329
Universal				
	665	227	712	294

The test of the Lehigh is an average of five briquettes in each case while that of the Universal is an average of fifteen briquettes.

Fineness Test.

Universal	Screen No.	Per Cent Passing
	75	98.9
	100	96.5
	200	82.5

Men skilled in mixing concrete were employed and all of the concrete in the beams was made in the proportions of 1 of cement, 2 of sand, and 4 of stone, measured by loose volume. The mixing was done by shovels. The sand and cement were first mixed together dry being turned over several times. The previously moistened stone was then added and the ingredients thoroughly mixed. Water was then added and the whole was turned until a fairly wet consistency was obtained as in this condition it could be tamped into the forms to much better advantage.

The steel used was ordinary commercial one-half in.

and three-eighths in. round bars with an elastic limit of about 40000 lbs. per sq. in. These bars were all straight except in four of the beams of Series II where they were bent up at an angle of 60 degrees.

5. TEST SPECIMENS.

Two types of test specimens were made(a) Cubes and (b) Beams.

a. The cubes were all of a 6 in. base. There were 3 cubes from each batch. As these cubes were not tested at the time that this thesis was writtn the results of the tests can not be given. The concrete was well tamped into the forms and troweled around the edges to insure a good surface to the concrete. The cubes were stored in dampened sand until tested. To insure plane surfaces the top and bottom faces are covered by plastic Plaster of Paris and levelled off by means of plate glass.

b.Two types of beams were used.

Series I. These beams had a cross section of 8 in. width and 10 in. depth to steel. Below the steel the width varied as shown in Table IV. The length in each case was 6 ft. 6 in.

Series II. All the beams of this series had a cross section of 14 in. width and 8 in. depth to steel. The total depth was 9 in. and the length 8 ft. in each case. The reinforcement is shown in Table IV.

6. FORMS for BEAMS.

The forms were made by laying planks on edge on the floor for the sides and ends. These planks were held in place by yokes of 2x4 plank and one-half in. bolts. The offsets in the bottoms of the beams of Series I were made by placing a piece of wood of the required shape in the lower edges of the forms.

7. MAKING of BEAMS.

The forms were soaked in water to prevent the wood from soaking up the water from the concrete and to decrease the tendency in the forms to warp. The concrete was mixed up as previously explained in paragraph 4 and poured into the forms until a layer 2 in. deep was on the bottom. The bars were then laid and the rest of the concrete put in and tamped at intervals. The top was then levelled off and the concrete allowed to set.

8. STORAGE of BEAMS.

The forms were usually removed from the cubes and beams at the same time after a period of seven days. The temperature of the laboratory was 60 to 65 degrees during the winter months but naturally increased as the warm weather began. The beams remained in a horizontal position until they were removed to the testing laboratory a few days before they were to be tested. The specimens were tested at the expiration of a period of from 58 to 63 days.

9. TESTING MACHINES USED.

For the first fourteen beams tested a vertical 2 screw Riehle machine of 100 000 lbs. capacity was used. This machine had 6 speeds of which the slowest, .05 in. per min. was used. The first five beams of Series II were tested in a Riehle machine of 600 000 lbs. capacity with a speed of .05 in. per min. The remaining beams of Series II were tested in an Olsen Machine of 200 000 lbs. capacity with a speed of .04 in. per min.

10. METHODS of SETTING SPECIMENS in MACHINES.

Series I. The beams of this series were all placed in the machine in the same manner. The beams were first raised by means of hoists and white-washed so that the cracks would be more distinct and then brought over the machine. The points of load application and support were then marked upon the beam. Roller bearings were placed under each end , the load being transferred from the beam to the roller by means of a bearing plate on which plaster of Paris had been placed before the beam was allowed to settle upon it. This allowed an even distribution of the load as the bottoms of the beams were somewhat uneven and varied from 4in. to 8 in. in width. The load points consisted of a spherical bearing block and a roller bearing. On these rested an I-beam to which the load was applied from the machine by means of a roller bearing planed on one side to render it more stable. The following drawing will show this more plainly than a mere description.

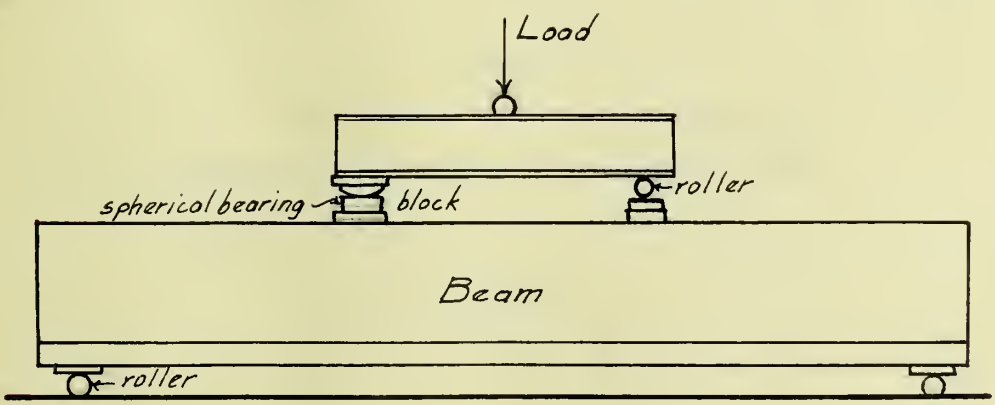


Figure 1.

Series II. The second series of beams was tested differently. In order to obtain readings of deformation in the reinforcing bars it was necessary to place the beams in the machine upside down. The beams were supported on the weighing table on two 4 in. rollers placed under the points of load. The load was applied on a seven and one-half foot span by means of cross I-beams to which the load was transferred from the machine by means of a larger I-beam. Between these I-beams bearing strips were inserted to take up any unevenness that might be in the beam or elsewhere. Cushions of fire hose were used under the I-beams and over the rollers to get an even distribution of the load. The first threebeams of this series were tested with the load points 12 in. apart, each 6 in. on either side of the center line. The last five Beams were tested with the loads at the one-third points.

The following drawing will illustrate this plainly.

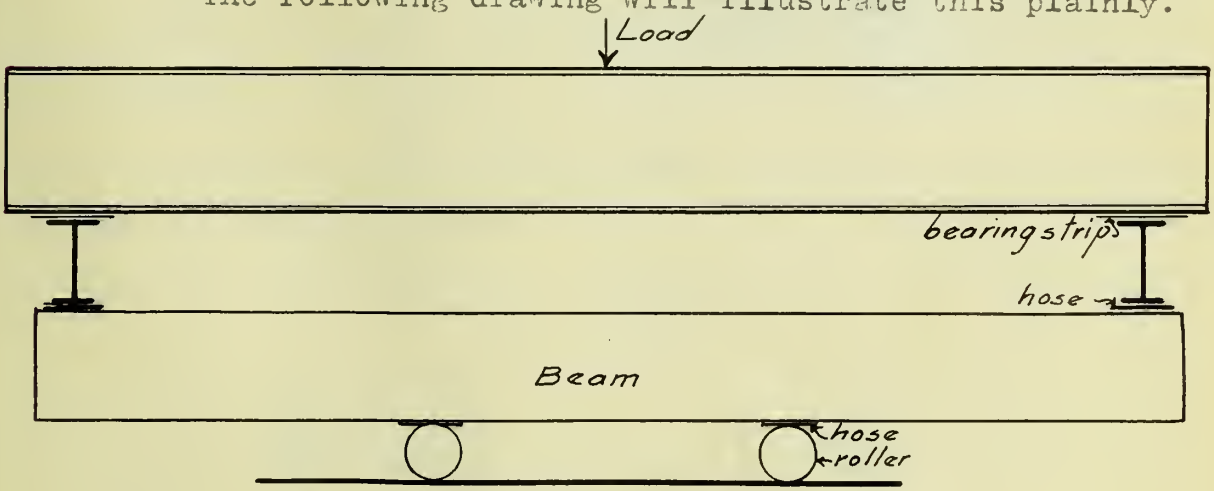


Figure 2.

11. MEASURING DEVICES.

No deformations were measured in the first set of beams except that of deflection. The deflections were read from a deflectometer (reading by interpolation to 0.0001 in.) affixed to a strip of wood one and one-half by two in. in size that was supported over the supports of the beams on conical points by means of clamps. The deflections read were not maximum as the deflectometer was but 2 ft. 8 in. from the support instead of 3 ft. This was necessary as the screw of the machine made the center of the beam inaccessible.

For the second series of beams the deformation in the bars was measured by means of the Berry extensometer. Holes three sixty-fourths in. in diameter were drilled in the bars at an equal distance on either side of the center line of the beam The deformations were then read at load intervals varying from 5000 lbs. to 3000 lbs. until the elastic limit of the steel had been passed.

The following drawing will illustrate the principle of the Berry extensometer.

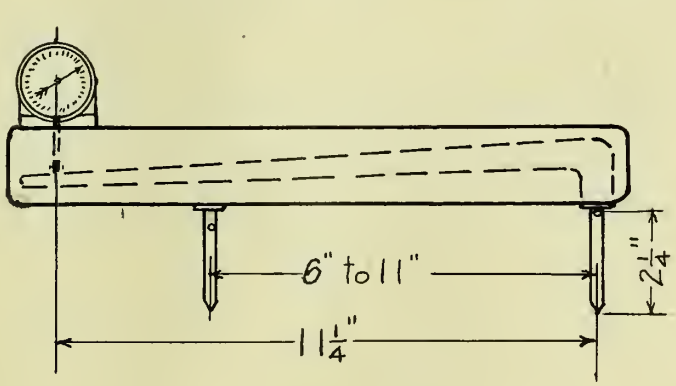


Figure 3.

12. APPLICATION OF LOAD.

In the first series of beams the load was applied in 2000 lb. increments until the first crack appeared when the increments were 1000 lbs. until failure. The machine was stopped at each increment as the writer was working alone and it was necessary to take the deflections, watch for cracks and run the machine. The load was applied at the one-third points on all of the beams of this series.

In the second series the increments varied from 5000 lbs. to 3000 lbs. growing smaller as the elastic limit was approached . At the end of each increment the machine was stopped, the deformations were taken and the beam carefully examined for cracks. After the elastic limit of the steel was passed the load was applied without stopping until the maximum load was obtained. On the first three beams of this series the load points were 12 in. apart but as it was observed that there was little variation in the stresses in the bars it was decided to load the remaining five beams at the one-third points. Deflections were taken on but one beam of this series as the apparatus was not ready until three beams had been tested on the Riehle machine and when the Olsen machine was used there was not sufficient room between the screws of the machine to use the apparatus.

III. EXPERIMENTAL DATA and DISCUSSION.

Beam 381.1			Beam 381.2		
Load	Def.	Remarks	Load	Def.	Remarks
2000	0.0053	Age 62 d.	2000	0.0038	Age 62 d.
4000	0.0125		4000	0.0086	
6300	0.0213		6000	0.0148	
8000	0.0287		8000	0.0224	
10000	0.0415		10000	0.0315	
12000	0.050	Crack	12000	0.0418	Crack
14000	0.062		14000	0.0525	Crack
16000	0.0778		160000	0.0642	Crack
18000	0.0928		18000	0.0780	Cracks
20000	0.129		20000	0.0945	
21000	0.168	Failed, beam read 11200	20800		Failed

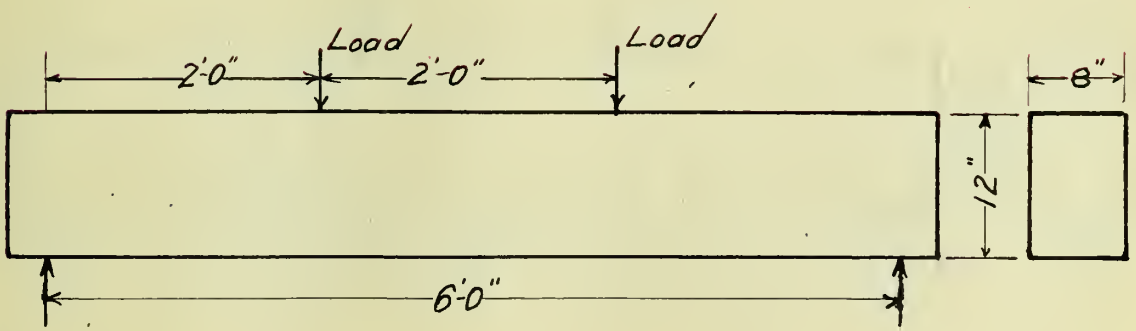


Figure 4.

Beam 382.1

Load	Def.	Remarks
2000	0.0040	Age 61 d.
4000	0.0098	
6000	0.0164	
8000	0.0245	
10000	0.0340	
12000	0.0440	
14000	0.0540	Crack
16000	0.0690	Cracks
18000	0.0830	
19700		Failed, beam read 8100

Beam 382.2

Load	Def.	Remarks
2000	0.0041	Age 62 d.
4000	0.0092	
6000	0.0150	
8000	0.0210	
10000	0.0295	
12000	0.0395	
13000	0.0455	Crack
13800		Failed, beam read 5000

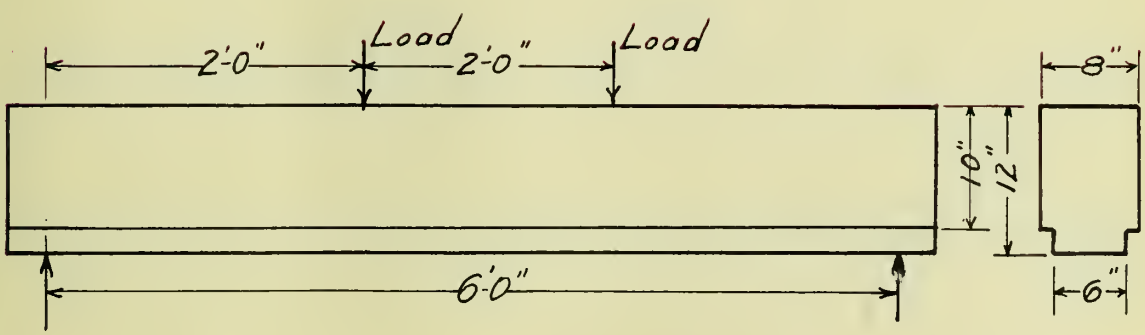


Figure 5.

Beam 383.1

Load	Deflection	Remarks
660	0.001	Age 62 days
2000	0.005	
4000	0.011	
6000	0.018	
8000	0.027	
10000	0.040	Crack
11000		Failed.
		Load on
		beam 3880.

Beam 383.2

Load	Deflection	Remarks
2000	0.005	Age 60 d
4000	0.010	
6000	0.018	
8000	0.027	
10000	0.037	
12000	0.049	Crack
14000	0.051	Crack
16000	0.076	Crack
17000	0.094	Failed

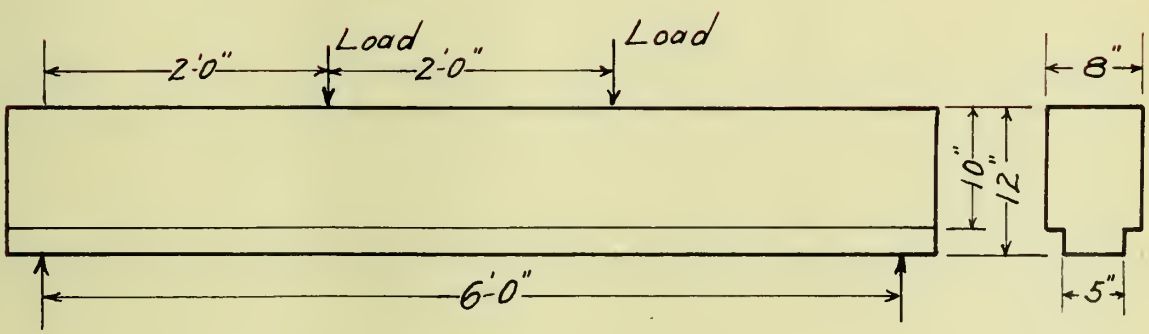


Figure 6.

Beam 384.1			Beam 384.2		
Load	Def.	Remarks	Load	Def.	Remarks
2000	0.006	Age 61 d.	2000	0.0058	Age 61 d.
4000	0.0165		4000	0.0120	
6000	0.0300		6000	0.0220	
8000	0.0458		8000	0.0358	
10000	0.0630	Crack	10000	0.0500	Crack
12000	0.0935	Cracks	12000	0.0660	
13000	0.0980		14200	0.0830	Crack
13900		Failed, beam	16000	0.1330	
		read 2400	18000	0.1270	
			19200		Failed

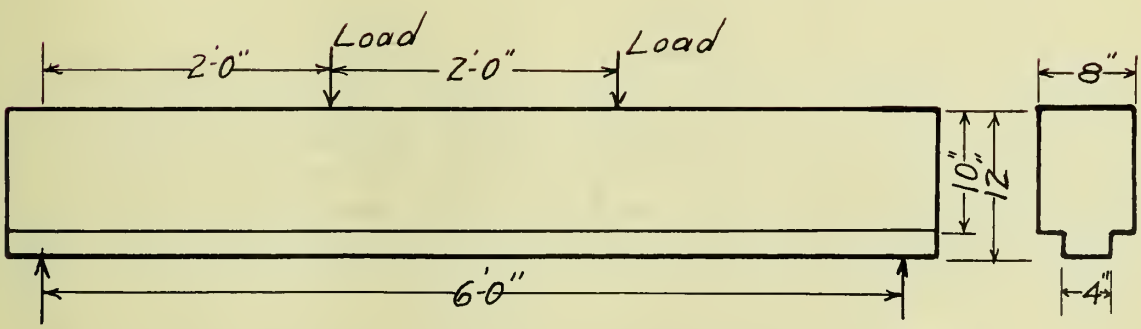


Figure 7.

Beam 385.1			Beam 385.2		
Load	Def.	Remarks	Load	Def.	Remarks
1100	0.0018	Age 64 d.	2000	0.0050	Age 61 d.
2000	0.0042		4000	0.0082	
4000	0.0135		6000	0.0174	
6000	0.0214		8000	0.0274	
8000	0.0320	Crack	10000	0.0380	
10000	0.0460		12000	0.0510	Crack
12000	0.0600		14000	0.0662	
14000	0.0765		16000	0.0810	Cracks
16200	0.0930		17700	0.1040	
17500		Failed, beam read 6500	19000	0.1250	
			19200		Failed, beam read 8200

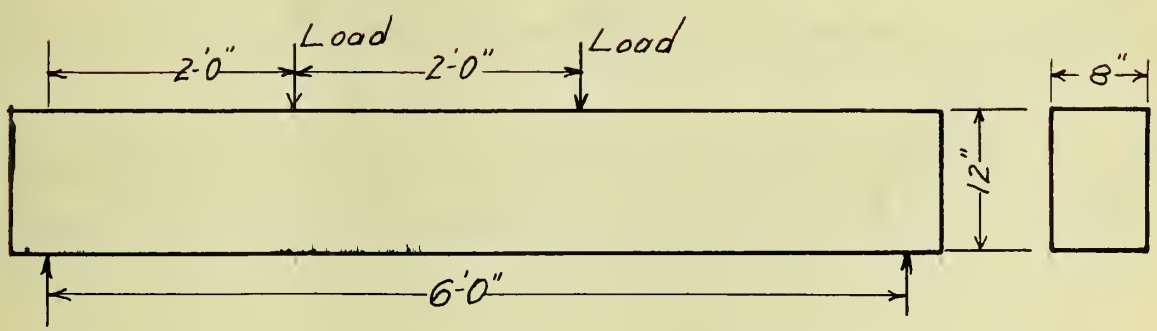


Figure 8.

Beam 386.1			Beam 386.2		
Load	Def.	Remarks	Load	Def.	Remarks
2000	0.0045	Age 62 d.	2000	0.0040	Age 59 d.
4000	0.0100		4000	0.0078	
6000	0.0162		6000	0.0110	
8000	0.0250		8000	0.0254	
10000	0.0368		10000	0.0358	Crack
12000	0.0505	Crack	12000	0.0530	Cracks
14000	0.0680		13000	0.0600	
14800		Failed, beam read 1200	13300		Failed, beam read 3600

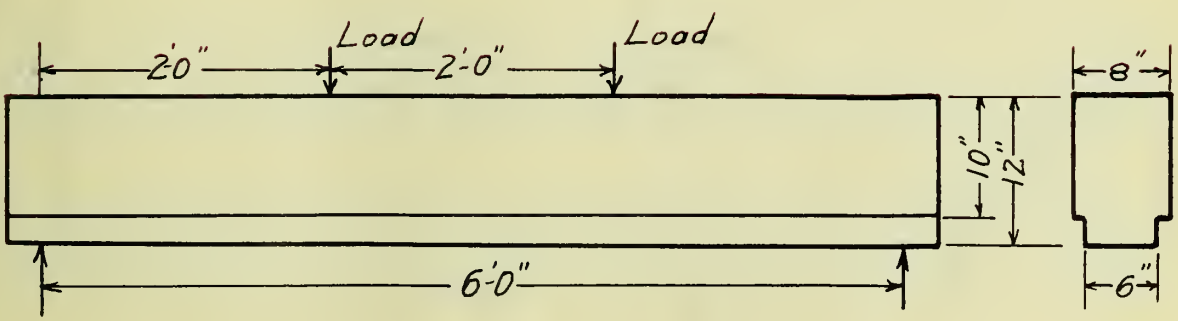


Figure 9.

Beam 387.1			Beam 387.2		
Load	Def.	Remarks	Load	Def.	Remarks
200	0.0000	Age 63 d.	2000	0.0040	Age 59 d.
2000	0.0056		4000	0.0078	
4000	0.0120		6000	0.0150	
6000	0.0270		8000	0.0242	
8000	0.0340	Crack	10400	0.0350	
9600	0.0500	Failed,	12300	0.0440	
		beam read	14000	0.0540	
		3200	16000	0.0630	Crack
			17000	0.0760	
			19400		Failed

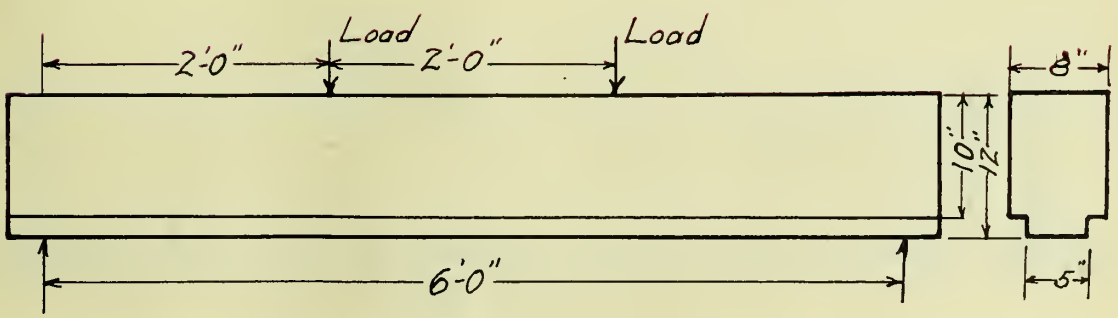


Figure 10.

Beam 388.1

Unit Stress in Bars.

Load	N 1	2	3	4	5	6	7
5000	6750	8250	7500	7500	7500	7500	8250
8000	11250	15000	14250	12000	13500	12750	13500
11500	19500	22500	21750	19500	21000	21000	20750
15000	27000	31500	28500	28500	28500	28500	28500
18000	33750	39000	36250	36250	40500	35250	36750

20800 Maximum load.

First crack at 8800 lbs.

Gage length 8 in.

Load points 6 in. on either side of center line.

Age 58 days.

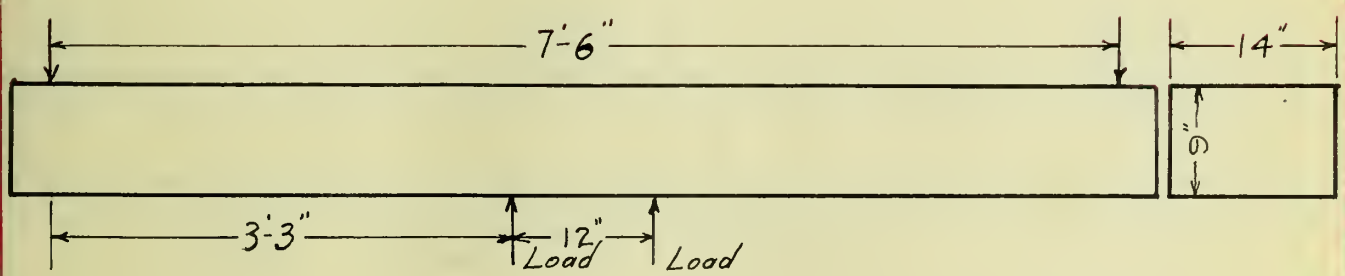


Figure 11.

Beam 388.2

Unit Stress in Bars.

Load	E 1	2	3	4	5	6	7	8
5000	4500	6750	7500	6000	9750	6000	6750	6750
10000	11250	9000	12000	13500				11250
13000	17250	16500	18000	20250		16500		17250
16000	18750	21250	22500	23250		23250		24000
19000	25500	27000	30000	29250		29250		30250
22000	33000	30750	36500	36000	31500	36000	37500	36000
25000	37500	36750	42000	39000	41250	36750	42000	38250

27500 Maximum load.

First crack at 16000 lbs.

Gage length 8 in.

Load at one-third points.

Age 59 days.

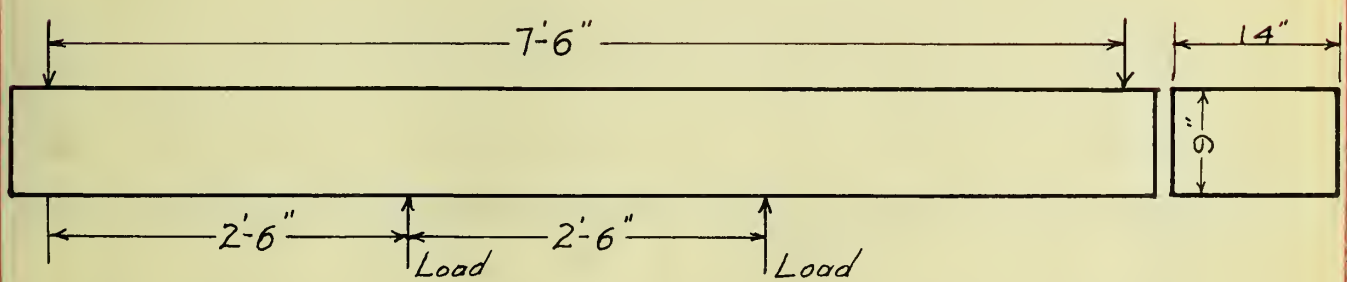


Figure 12.

Beam 388.3

Unit Stress in Bars.

Load	N 1	2	3	4	5	6	7	S 8
5000	12000	12000	12750	12750	13500	12750	12000	11250
8000	15000	15000	16500	18000	17250	16500	15000	15000
11500	27750	22500	24000	24000	22500	23750	21000	21000
15000	27750	29250	30750	28500	29250	29250	27750	27750
18000	37000	39000	41500	39000	37500	37500	35250	35250
20300	73500	82500	100250	74250	73250	60000	69000	63000
21600	Maximum load.							

Loads 6 in. on either side of center line.

Bars bent up.

First crack at 8000 lbs.

Gage length 8 in.

Age 56 days.

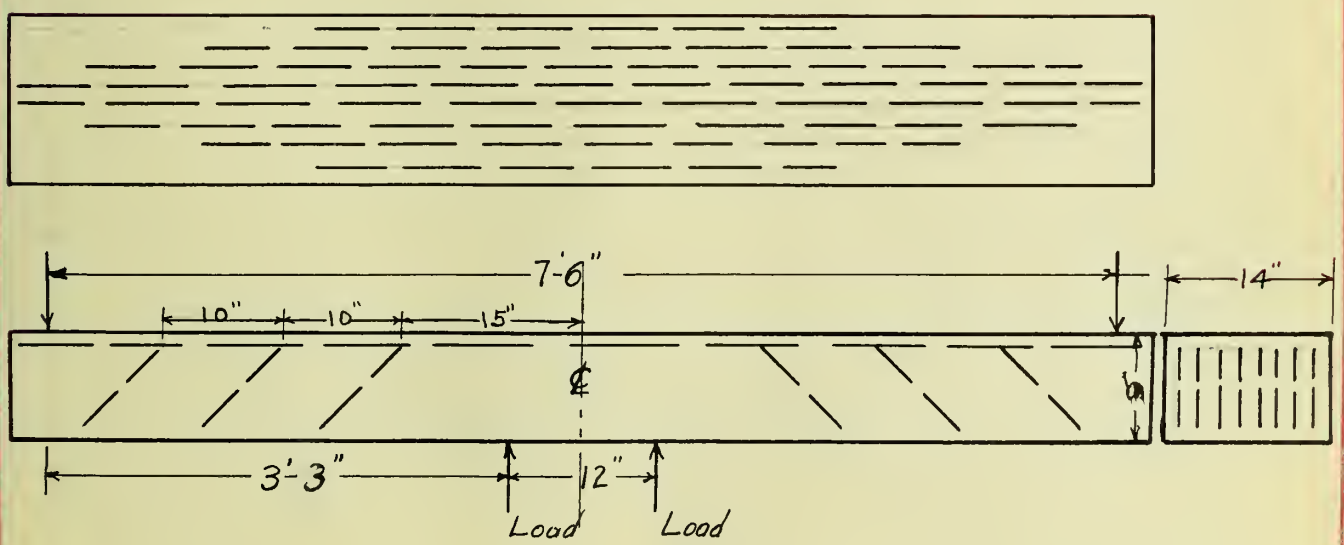


Figure 13.

Beam 388.4

Unit Stress in Bars.

Load	E 1	2	3	4	5	6	7	8
5000	4500	5250	6750	3750	7500	3750	3000	1500
10000	11250	10500	11250	6750	15500	12000	12750	13250
15000	18750	18000	16500	16500	24750	19500	19500	18000
18200	24750	24000	27000	24750	32750	27500	24000	25250
21000	28500	36750	32250	30000	36000	33750	28500	27750
24000	33750	42000	38250	32250	42000	36000	36000	32250
26700	Maximum load.							

Gage length 8 in.

Load at one-third points.

For diagram of reinforcement see the Summary Table.

Age 60 days.

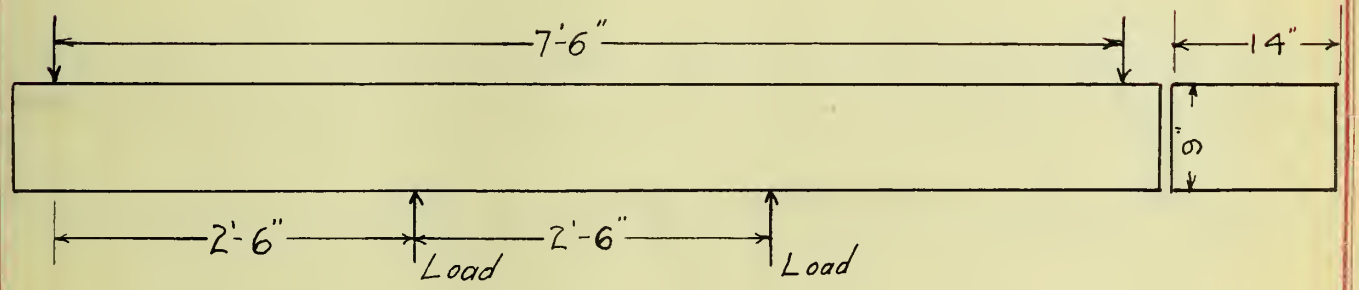


Figure 14.

Beam 389.1

Unit Stress in Bars.

Load	N 1	2	3	4	5	6	7	14
5000	8250	9750	12750	12000	12750	10500	11250	12750
8000	10500	11250			15000			12750
8000	12000	13500	16500	17250	15750	15000	15000	15000
11500	21000	22500	22500	21750	24000	22500	21750	24750
15000	27750	28500	30000	30750	32250	31500	32250	32250
18000	34500	36000	37500	39750	39750	39000	39750	39000

23100 Maximum load.

After a load of 8000 lbs was on the machine the readings were so uneven that the load was taken off and two new bearing strips were inserted between the I beams.

First crack at 7800 lbs.

Load points 6 in. on either side of the center line.
Gage length 8 in.

Age 57 days.

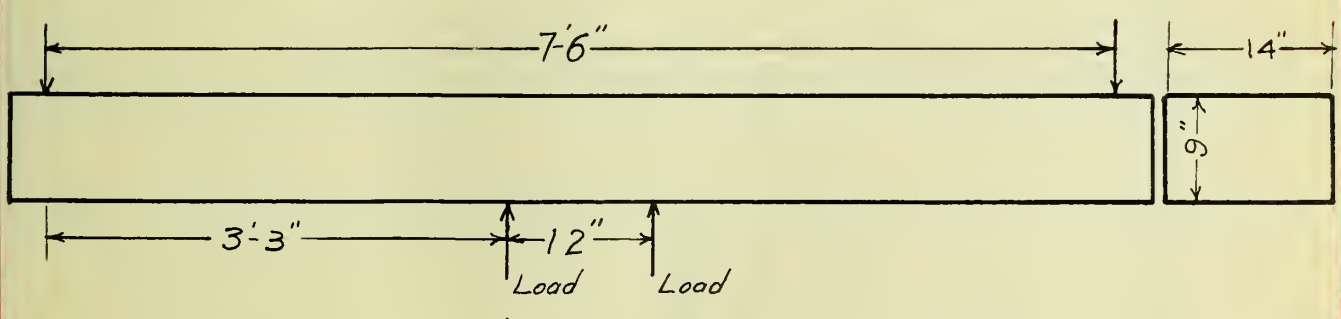


Figure 15.

Beam 389.2

Unit Stress in Bars.

Load	E 1	2	3	4	5	6	7	14
5000	4200	4700	3600	3600	4800	4800	4800	2400
10000	10200	9600	10200	10200	10200	10200	9600	9600
15000	19200	19800	20400	21600	19800	20400	19800	19200
20000	30000	30600	30600	30000	31200	30600	30600	

31900 Maximum load.

First crack at 14000 lbs.

Load at one-third points.

Gage length 10 in.

Age 61 days.

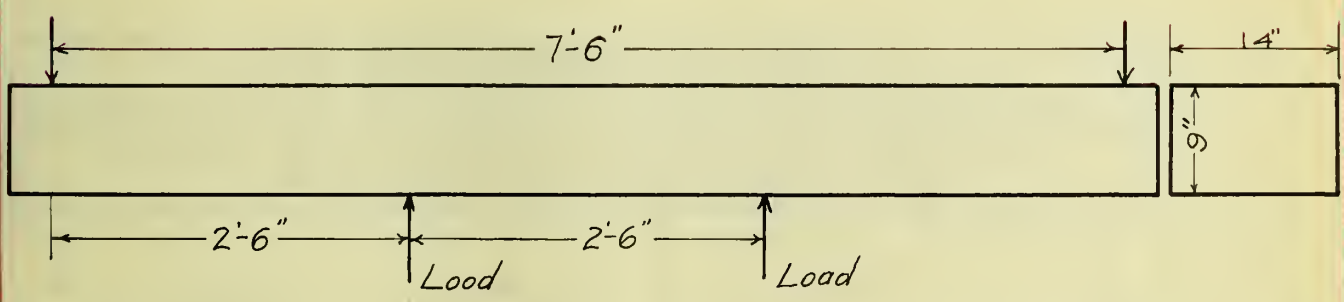


Figure 16.

Beam 389.3

Unit Stress in Bars.

Load	N 1	2	3	4	5	6	7	14
5000	3750	3750	4500	750	6750	5250	5250	3750
8000	8250	8250	8250	3000	9000	8250	9000	6750
11000	12750	11250	11250	6000	15000	13500	14250	9750
14000	19500	18000	18000	12750	21000	18000	18750	15000
17000	24750	22500	23250	18000	26250	24000	24000	19500
20000	30750	28500	26250	24000	30750	30000	28500	28250
23000	33000	30750	32250	28500	33750	34500	33750	25500
29000	Maximum load.							

Gage length 8 in.

Load points at one-third points on beam.

Age 60 days.

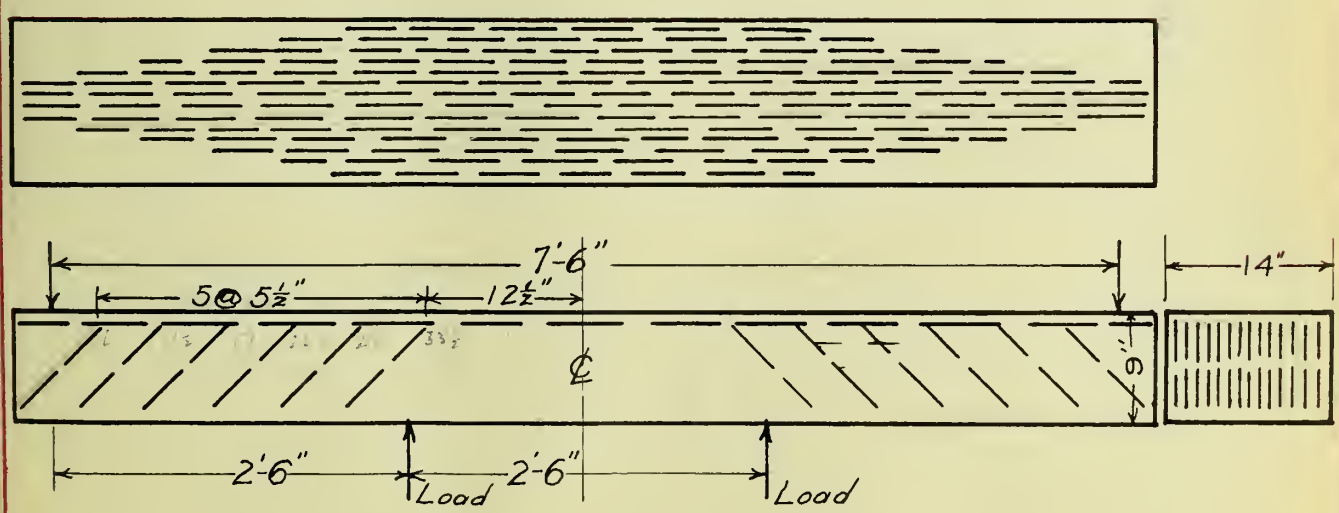


Figure 17.

Beam 389.4

Unit Stress in Bars.

Load	E 1	3	5	7	8	10	12	14
5000	6000	3000	4200	4800	4800	4800	3000	4800
10000	12000	9600	10200	10200	11400	9600	7200	10200
15000	21000	19800	19200	19800	21000		15600	19200
20000	30000	28800	30600	29400	30600		23400	28200
25000	36000	40200	40800	38400	40800		33000	36000
29500	Maximum load.							

First crack at 10000 lbs.

Load at one-third points.

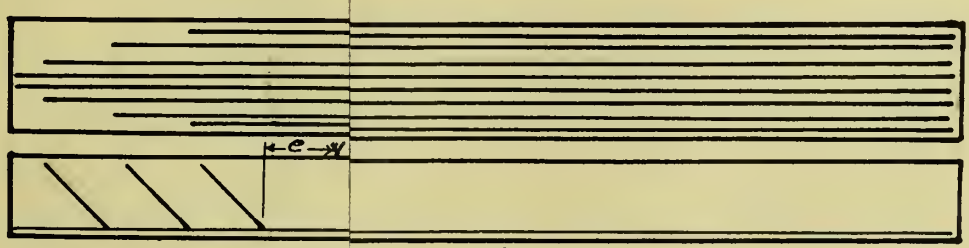
Gage length 10 in.

For diagram of reinforcement see the Summary Table.

Age 61 days.

Reference Number	Reinforcemen			hod of Failure.
	No of Bars	Size	C	
381.1	6	½	0.8	nal Tension
381.2				"
382.1	6	½	0.5	"
382.2				" and Horizontal Shear.
383.1	6	½	0.3	" " " "
383.2				ontal Shear
384.1	4	½	0.5	nal Tension
384.2				" and Horizontal Shear.
385.1	9	⅜	0.5	"
385.2				"
386.1	9	⅜	0.2	" " " "
386.2				ntal Shear
387.1	9	⅜	0.1	"
387.2				"

Reference Number	Reinforcment			L.		Maximum Bond	Remarks
	No of Bars	Size	Typ		Calculated		
388.1	8	½	C		44000	140	
388.2			C			183	Failed completely.
388.3			B	"		540	
388.4			A	"		580	Maximum bond on short bar.
389.1	14	⅜	C		41000	98	
389.2			C			124	Concrete crushed
389.3			B	.5"		405	
389.4			A			405	



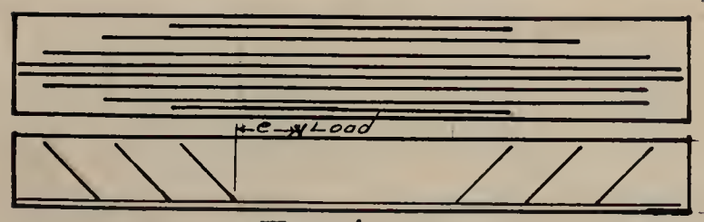
Typ Type C

Summary Table. Series I

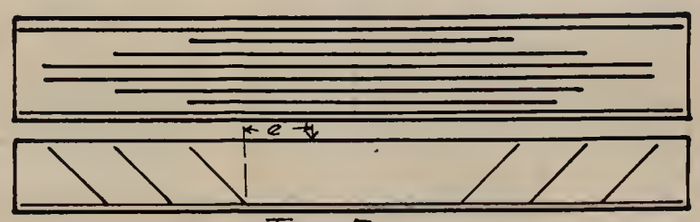
Reference Number	Reinforcement			Date Tested	Age	Maximum Load	Unit Shearing Stress		Bond	Stress in Steel	Width at Steel	Method of Failure.
	No of Bars	Size	c				Vertical	Horizontal				
381.1	6	½	0.833	3-22-11	62	21000	131	130	140	31600	8	Diagonal Tension
381.2				4-12-11	62	20800	130	131	139	31400		" "
382.1	6	½	0.50	4-5-11	61	19700	123	207	132	29800	6	" "
382.2				4-21-11	62	13800	86	148	93	20880		" " and Horizontal Shear.
383.1	6	½	0.333	3-22-11	62	11000	69	173	73	16600	5	" " " " "
383.2				4-19-11	60	17000	106	266	113	25600		Horizontal Shear
384.1	4	½	0.50	4-5-11	61	13900	87	103	131	19800	4	Diagonal Tension
384.2				5-1-11	61	19200	120	143	182	27400		" " and Horizontal Shear.
385.1	9	⅜	0.512	3-24-11	64	17500	109	115	100	25500	8	" "
385.2				5-1-11	61	19200	120	127	110	27900		" "
386.1	9	⅜	0.291	4-12-11	62	14800	94	172	85	21600	6	" " " " "
386.2				4-29-11	59	13300	83	154	76	19400		Horizontal Shear
387.1	9	⅜	0.18	3-23-11	63	9600	60	170	52	13900	5	" "
387.2				4-29-11	59	19400	121	364	111	28200		" "

Series II.

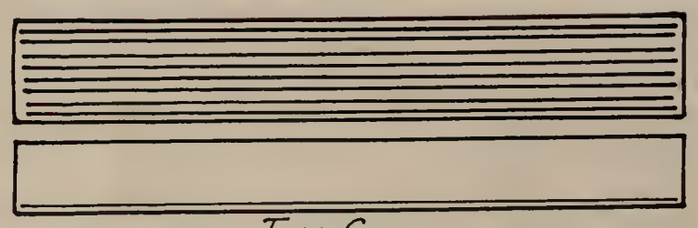
Reference Number.	Reinforcment			Date Tested	Age	Maximum Load	Vertical Shear	Distance between Loads	Resisting Moment	Tension in Steel.					Maximum Bond	Remarks
	No of Bars	Size	Type							Measured Maximum	e	Measured Minimum	e	Calculated		
388.1	8	½	C	5-13-11	58	20800	93	12"	405000	5. 40500	0	1 33750	0	44000	140	
388.2			C	5-18-11	59	27500	123	30	412000	3.7. 42000	0	2.6. 36750	0		183	Failed completely.
388.3			B	5-11-11	56	21600	96	12"	422000	3. 41500	19"	7.8. 35250	9"		540	
388.4			A	5-19-11	60	26700	119	30	401000	2.5. 42000	10"	4.6. 32250	0"		580	Maximum bond on short bar.
389.1	14	⅜	C	5-12-11	57	23100	103	12"	451000	4.5.7. 39750	0	1 34500	0	41000	98	
389.2			C	5-20-11	61	31200	140	30	468000	5. 31200	0	1. 30000	0		124	Concrete crushed
389.3			B	5-15-11	60	29000	130	30	436000	6 34500	28.5"	9. 18000	28.5"		405	
389.4			A	5-20-11	61	29500	132	30	442000	8.8 40800	19.5"	12. 33000	8		405	



Type A



Type B


Type C

13. TABLES and DIAGRAMS.

Series I. In the Summary Table is condensed all of the important data of the beams. The vertical shear obtained is found by dividing the total shear by the cross section of the beam. The total stress in the steel was figured from the formula $S_s = M/jd$. The moment was in in. lbs. and j was obtained from the graph on pg. 279 in Turneaure and Maurer for the several per centages of steel in the beams. The horizontal shear was obtained from the formula $v' = uo/2c$ where u is the unit bond stress, o the circumference of the bar and c is the average clear space between the bars. The average unit bond stress u was obtained from the formula $u = S/24no$ where S is the total stress in the steel, n is the number of bars and the other letters stand for the same quantities as above.

Figures 18 and 19 give the deflection curves for all of the beams. Figure 20 ,a diagram, shows the relation between the space between the bars and the vertical and horizontal shearing stresses. These will be discussed later. Figures4 to 10 inclusive show the manner of loading and cross sections of the beams.

Series II. The manner in which the reinforcing bars were placed in this series is shown on the Summary sheet. Figures 11 to 17 inclusive are curves showing the relation between the stress in the several bars and the load. Figures 22 to 29 inclusive show the relation between the stresses in all of the bars in the beam at the same load. Figure 34 shows the variation of the unit bond stress in the different bars for

the beams that had their bars bent up. It also shows the variation of the maximum unit bond stress in the same bars. The maximum stress would be in the part that is bent up. Figures 11 to 17 inclusive show the method of loading the beams and the manner in which the bars are placed when they are not straight.

Stresses of these diagrams were computed from the readings of the Berry extensometer. This instrument could be depended upon to detect a stress of 750 lbs. per sq. in. in the steel on a gauge length of 8 in. and of 600 lbs. per sq. in. on a gauge length of 10 in.

In computing the bond stress in the Summary Table the full length of the bent up part of the bar was used. This length was $9\frac{1}{2}$ in. on the average.

14. PHENOMENA of TESTS.

Series I. As is usual in such tests small vertical cracks from 1 in. to 3 in. long appeared under the load points or at a short distance to one side or the other of them. These cracks became longer as the load was increased but seldom exceeded 4 in. or 5 in. in length. As the load was increased new cracks appeared towards the supports and these cracks became more inclined to the horizontal the farther away they were from the load points. In many of the beams having a reduced width at the plane of the steel a crack forming at some point on this plane extended upward toward the load point and at almost the same time horizontally along the fillet toward the support. The manner and way.in which the cracks appeared can be better obtained from the photographs than from a mere reading of the description of them.

The first small cracks appeared at a load ranging from 30 per cent to 90 per cent of the ultimate load although most of the beams begam cracking at a load of about 60 per cent of the ultimate load. In bea 383.1 the first crack appeared at 10000 lbs. and the beam failed at 11000 lbs. This beam evidently failed in horizontal shear as the crack along the fillet was about 10 in. long and the concrete separated from the bars on the lower side. Probably the best example of a failure by horizontal shear was that of beam 386.2. In this beam the failure was almost vertical and inside of the load point. It then ran along the fillet to the south support stripping the entire third part of the concrete below the bars away from them.

The most abnormal test of the series in the writer's estimation was that of 387.2. This beam carried over twice the load that the one similar to it carried and almost as much as the beams that were expected to carry the most. The concrete must have been exceptionally well mixed and tamped about the bars. This beam broke with quite a smart shock. The first crack did not appear until after a greater load had been applied than had broken the one like it.

Series II. On beams 388.3; 389.1; 388.1; the loads were applied 6 in. on either side of the center line of the beam. The first cracks were visible on these three beams at about 8000 lbs.load. The cracks were small and on the sides of the beams. Very soon afterwards cracks appeared on the top (the surface nearest the reinforcement) almost over the support. As the load was increased more cracks appeared, these coming one after the other towards the support and being very

nearly at right angles to the sides of the beams. When the ultimate load was nearly reached more cracks appeared among the first ones. On the beams loaded at the one-third points the first cracks did not appear until from 14000 lbs. to 19000 lbs was on the machine. The sequence of their appearance was the same as described above though there were not nearly as many cracks altogether.

In the beams which had the bars running straight through, the cracks on the sides were the same as those in the usual beam unreinforced for web stresses being vertical over the supports and making smaller angles with the horizontal as they were farther away from the load point. On the beams with the bars bent up the cracks were all nearly vertical having very little inclination. The cracks on the tops of the beams invariably began at the edges and worked across to the opposite edge or began at the two edges and met in the inside of the beam. Those beams having the bars running straight through had the cracks on the top run all of the way across while those with part of the bars bent up had some of the cracks end before extending all of the way over, usually stopping where the bend in the bar occurred.

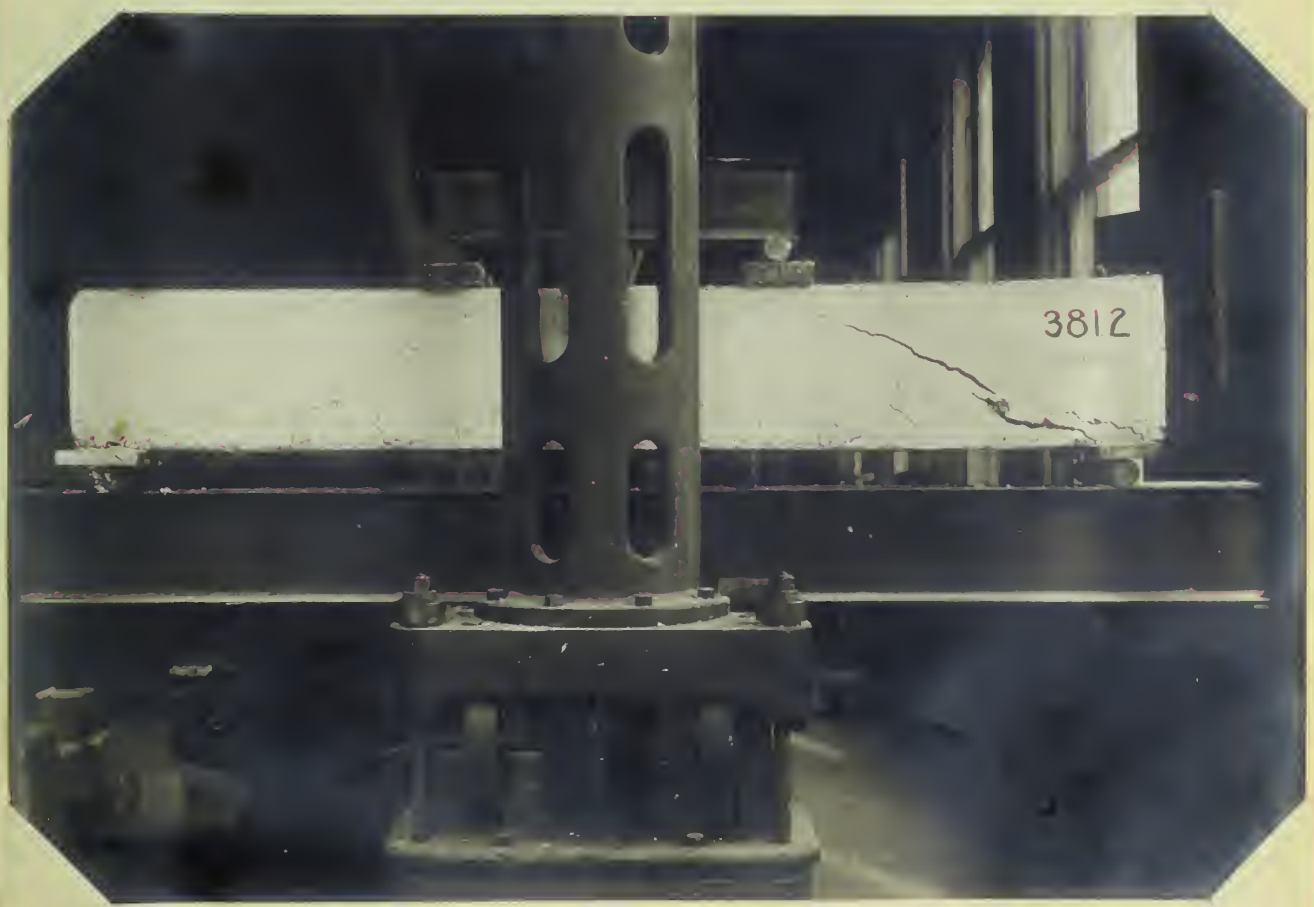
15. MANNER of FAILURE.

Series I. (a) Beams reinforced with 1/2 in. bars.

The manner of failure in the first set of beams was either diagonal tension, horizontal shear or a combination of the two. The best idea of failure can be obtained from the photographs.

Beam No. 381.1 plainly failed by diagonal tension as

3811
21000#


3812

382.1
19700#

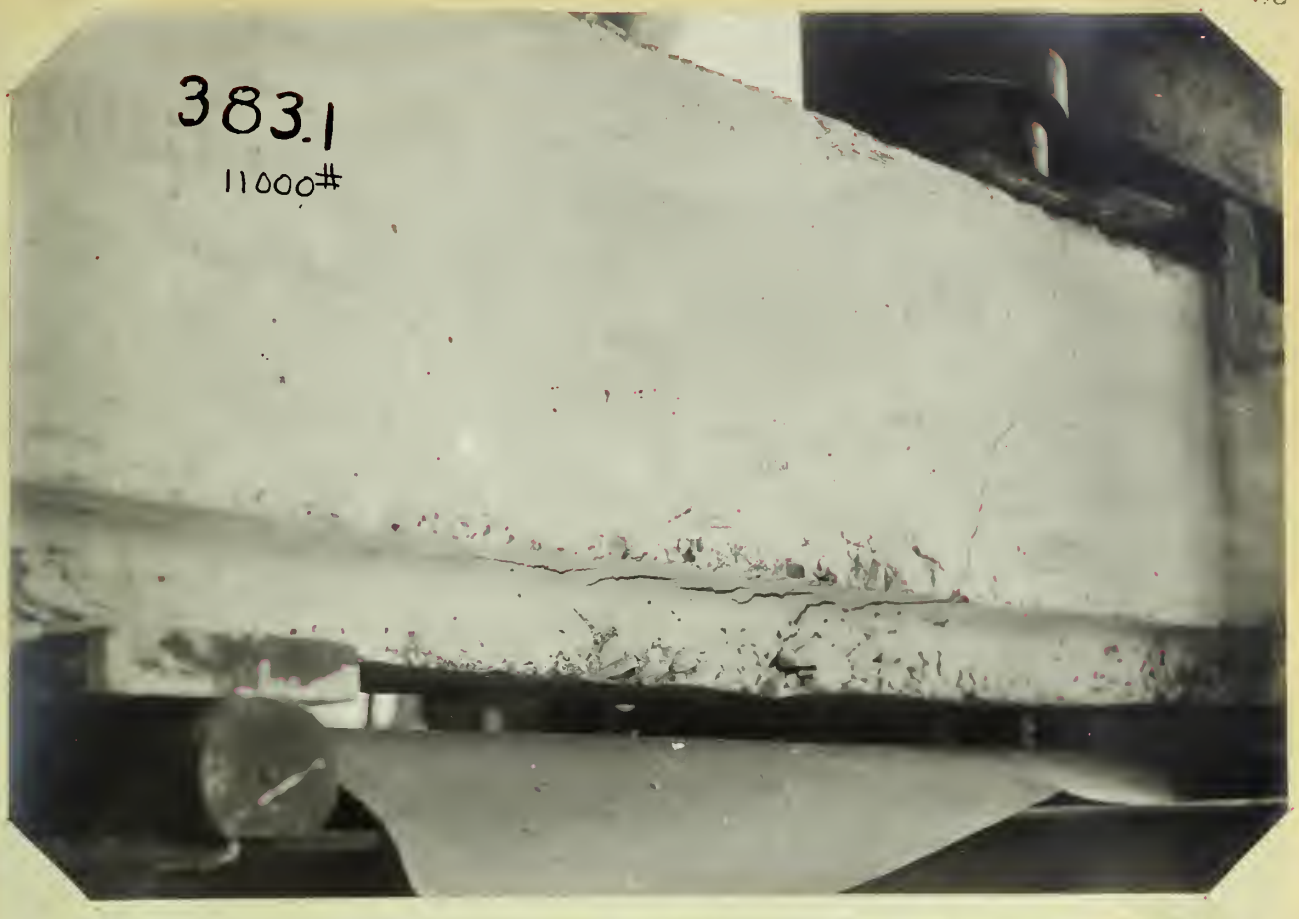
382.2
13600#

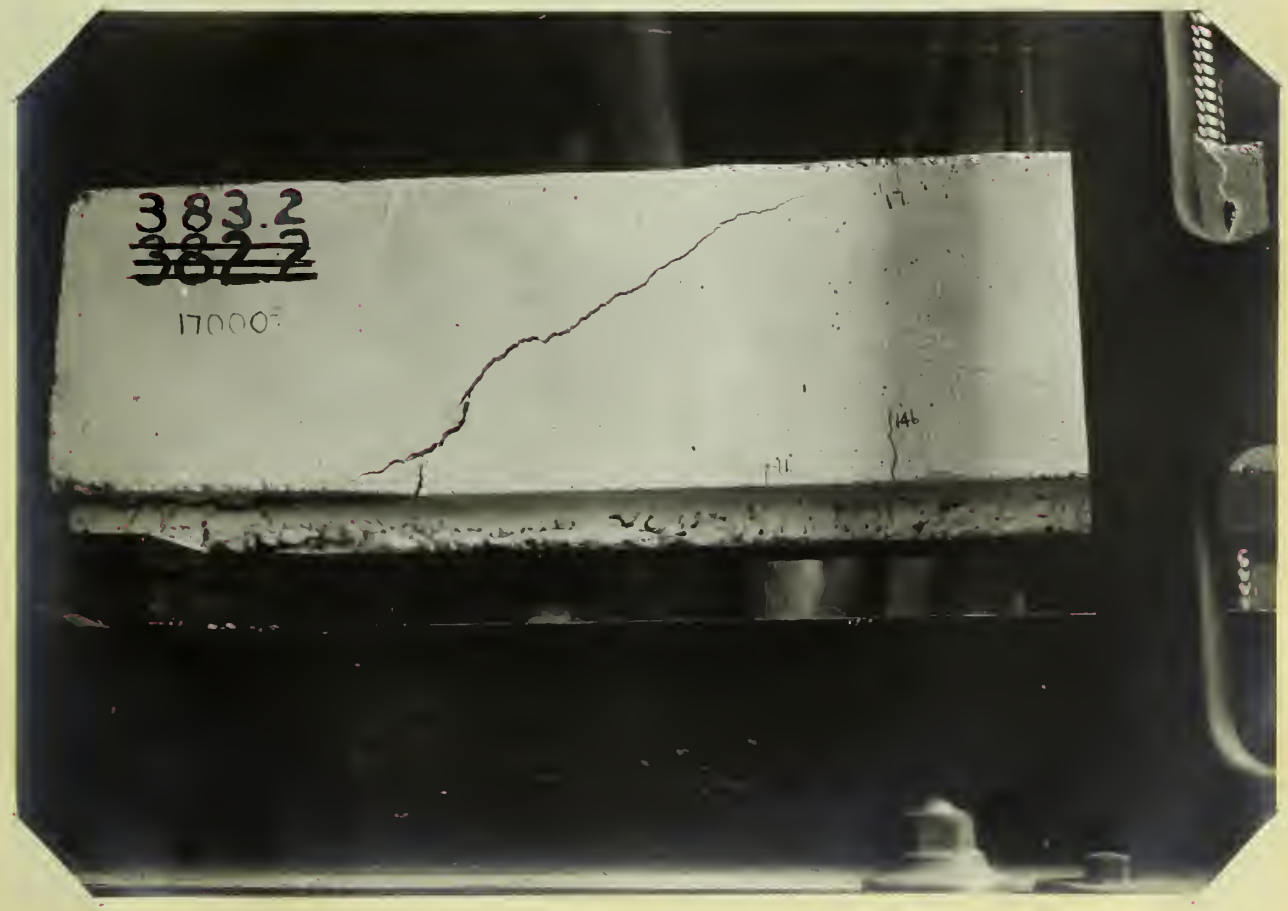
can be seen from the cracks running from the load points to the supports in almost straight lines. the bars slipped at one end but whether this occurred before or at failure is not known. This beam would hardly be expected to fail by horizontal shear as the clear distance between bars was one and two-thirds times the diameter of the bar. Beam No. 381.2 failed in almost the same manner though there were a number of smaller tension cracks. These failures are typical for beams without web reinforcement.

In Beam No. 382.1 the failure might be that of horizontal shear but in the writer's estimation was principally from diagonal tension. The weighing beam dropped at an ultimate load of 19000 lbs. and the load fell off to 8100 lbs. If the failure had been by horizontal shear the bars would have become loosened and then they would have slipped as they did in some of the later cases. In such a case the beam would probably have failed completely and the load have fallen off to zero. Beam No. 382.2 failed in the same manner although the crack along the fillet was much longer than beam No. 382.1. In Beam No. 382.2 horizontal shear entered into the failure to some extent probably, as neither the ultimate load nor the load on the beam after it had failed was as great as in No. 382.1. The low ultimate strength was probably due to the fact that the concrete below the bars at the south end was full of voids and that the outside bar was exposed for about half its circumference for a distance of perhaps 18 in. from the end of the beam.

Beam No. 383.1 failed in much the same manner as Beam No. 382.1 and the same conclusions will hold for it.

Beam No. 383.2, however, failed by horizontal shear

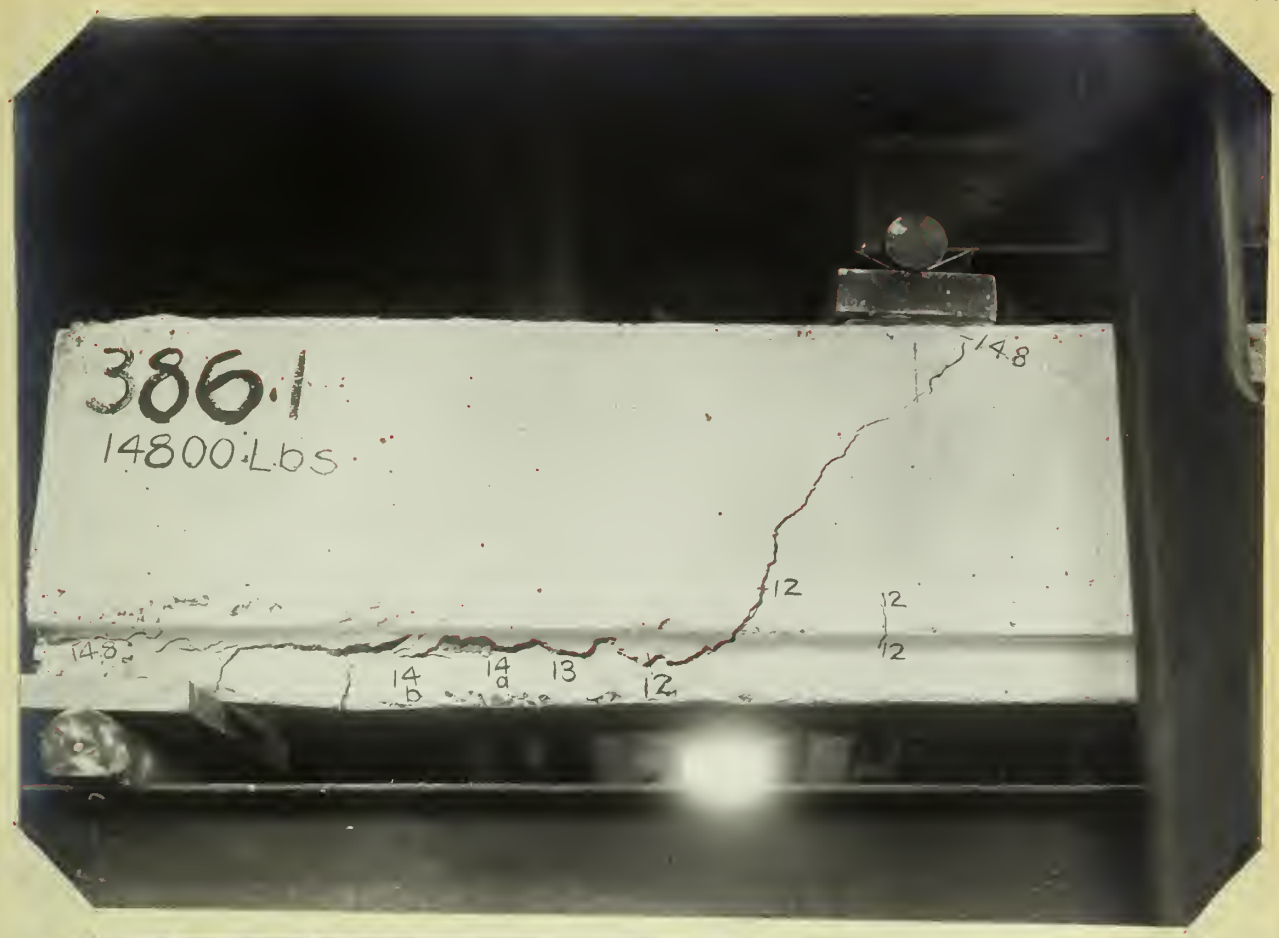

383.1
11000#


383.2
~~383.3~~
~~3822~~
17000#

19.2
384.2
19200#
SW
14
15.1
10
11

387.1
385.1
17500#
383.1
381.1
W
E
E

385.2
19200#
19
17
16.9
16
16
16
16
16


386.1
14800 Lbs
14.8
12
12
12
14.8
14 b
14 a
13
12

386.2
13300#
13.3

or a combination of horizontal shear and diagonal tension. This beam broke under a load of 17000 lbs. while the machine was not running and just after the deflection had been read. The failure was complete, the load falling off to zero. The bars slipped about 3/16 in. at the failure end, probably when the beam broke.

Beam No. 384.1 failed plainly by diagonal tension as can be seen from the photograph. The failure was not complete the load falling off to 2400 lbs.

Beam No. 384.2 probably failed by a combination of of the two stresses. The concrete in this beam was better than that of the previous one as could be told by looking at the beam and by the much higher ultimate load it carried. In this case the failure was complete, the concrete below the bars for 12 in. to the left of the south support cracking completely awayfrom the bars. Here the bars slipped about 3/16 in.

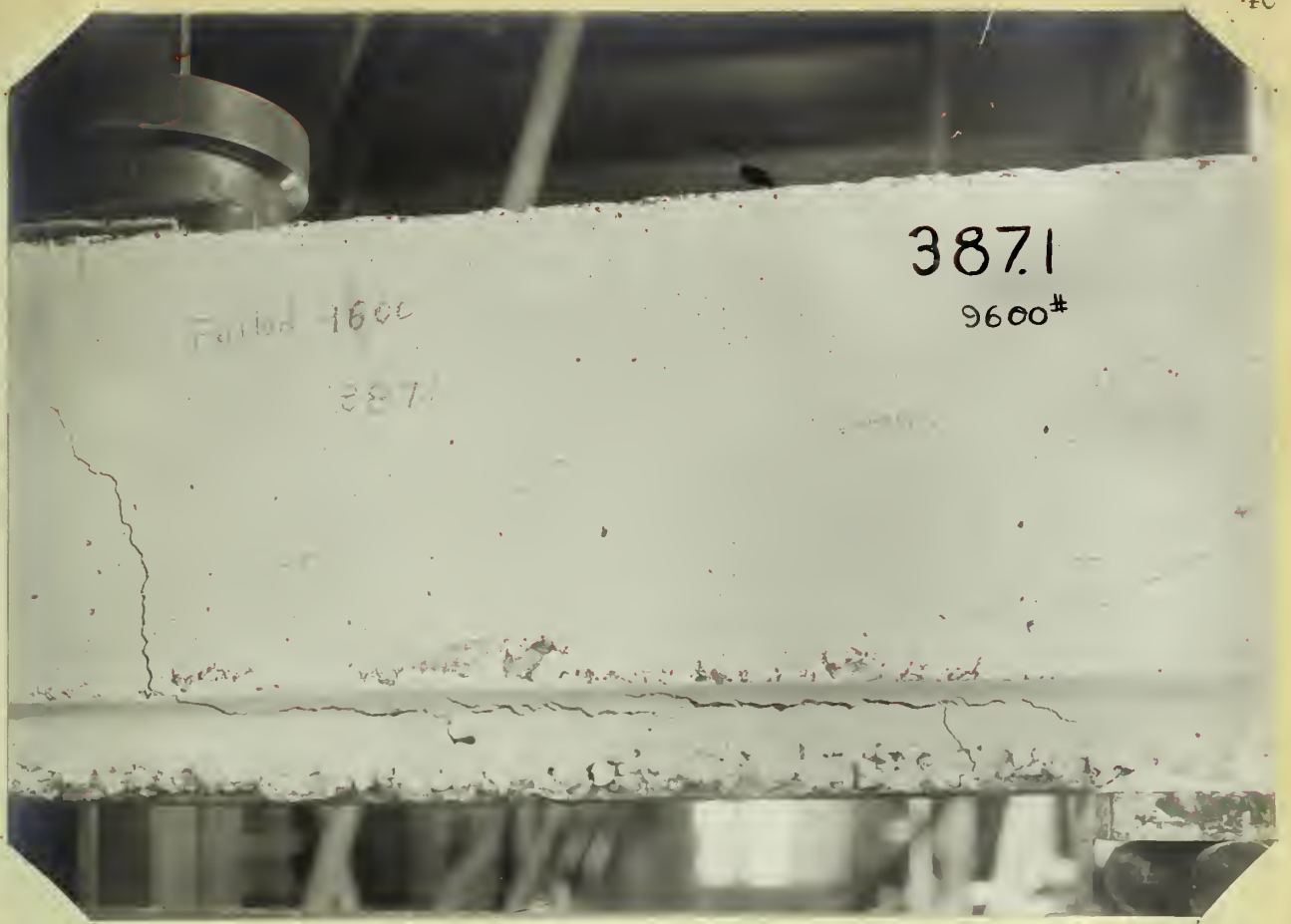
In Beam No. 385.1 the failure was by diagonal tension the load after failure reading 6500 lbs.

(b). Beams reinforced with 3/8 in. bars.

In Beam No. 385.2 the failure was much the same as that of the previous one. The concrete in the latter beam was of better quality than in the former as was indicated by the larger number of cracks and the larger ultimate load. The bars slipped about 1/16 in. in this case and the slipping may have been general as the bars came away from the concrete very easily when the beam was broken up.

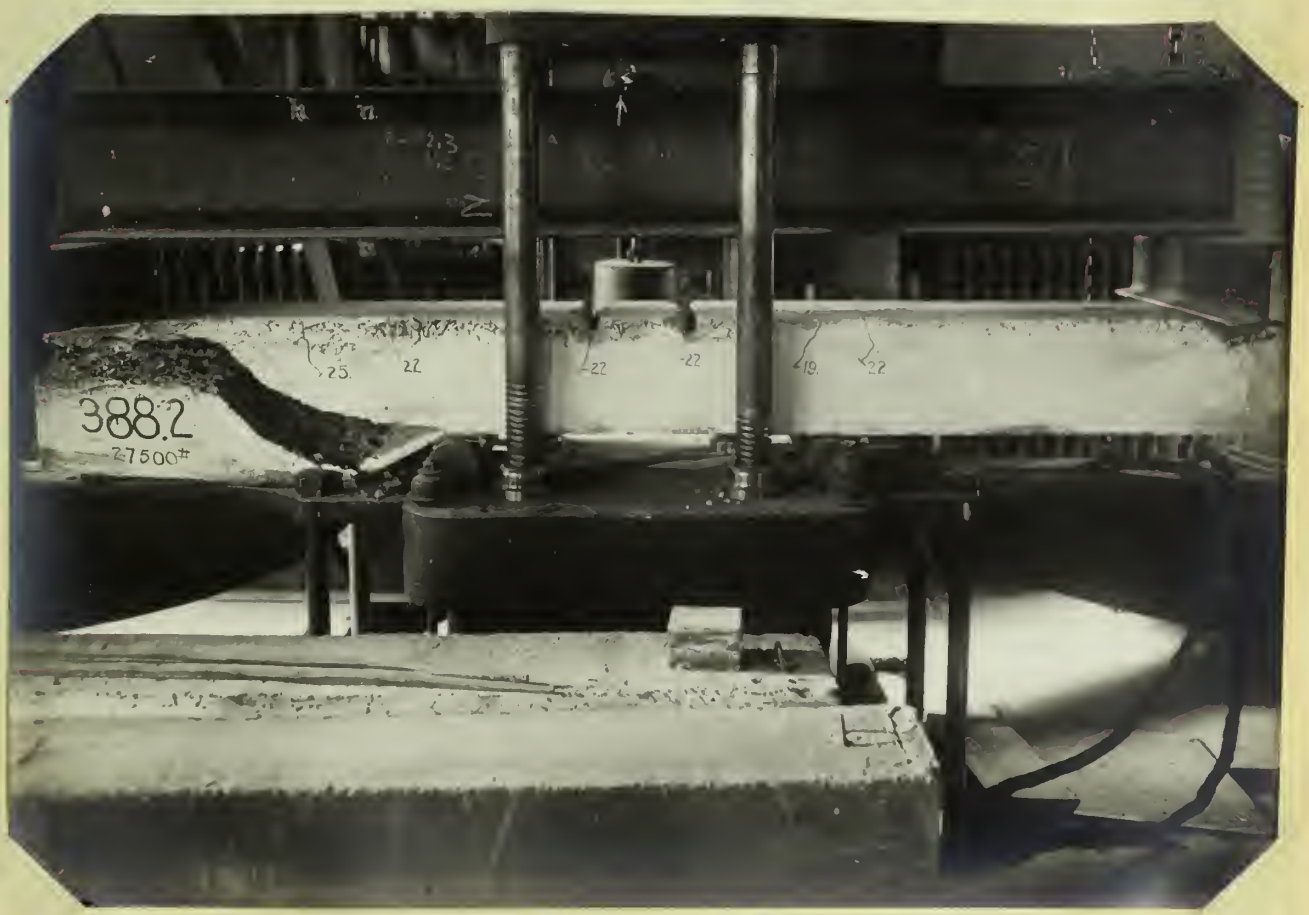
Beam No. 386.1 probably failed by a combination of horizontal shear and diagonal tension as it did not fail completely, though the crack along the fillet was quite extensive.

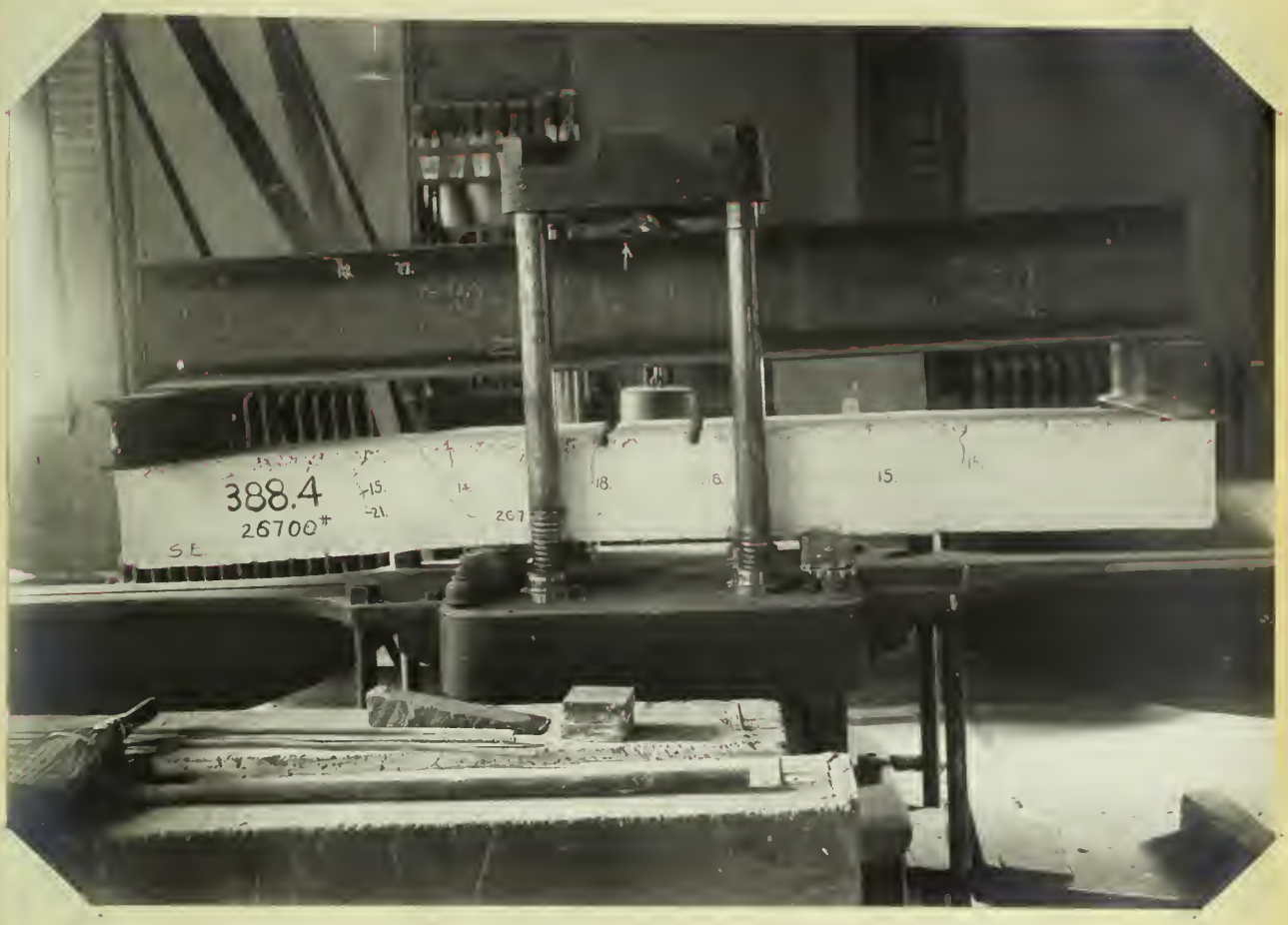
Beam No. 386.2 failed, evidently by horizontal shear


387.1
9600#

387.2
19400#
18.9

293.2
287.5
286.7
388.1
20800#


388.2
27500#


388.4
26700#
SE

as the crack ran almost verticaly downward from the south load to the south support. Though the load read 3600 lbs. after failure it was no doubt due to the bond between the upper part of the beam and the bars.

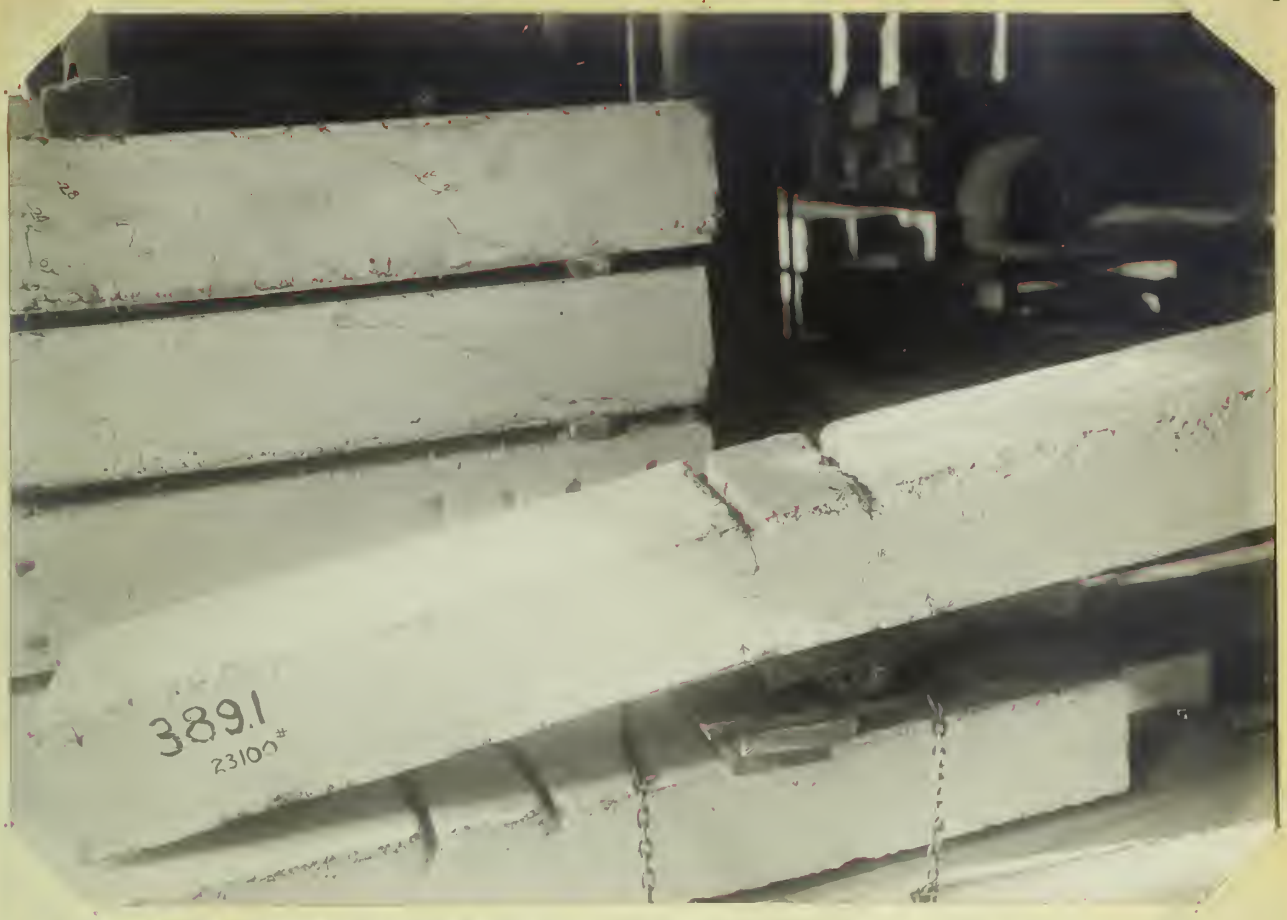
Beam No. 387.1 failed very nearly as did No. 386.2.

No. 387.2 carried more than twice what No. 397.1 did. The failure was complete and typical of the others. No cracks whatever appeared until 16000 lbs. load and the beam failed at a load of 19000 lbs. The beam failed with a much sharper concussion than most of the others. The bars had slipped 3/16 in. at the failure end and had broken loose entirely from both the upper and lower parts of the beam. The high load carried was probably due to the fact that the bars had been thoroughly surrounded by the concrete.

Series II. All of the beams failed in the same manner that is, by the stress in the steel going beyond the elastic limit before the concrete had failed. But one beam, namely, No. 388.2 failed completely. This particular beam broke in two parts, the line of fracture running from the south support to the south load point. The steel had failed previously. The other beams continued to carry their maximum loads for some time, the deflection becoming constantly greater. It was noticeable that those beams having the rods bent up held their maximum loads longer and the cracks were both fewer in number and smaller in size.

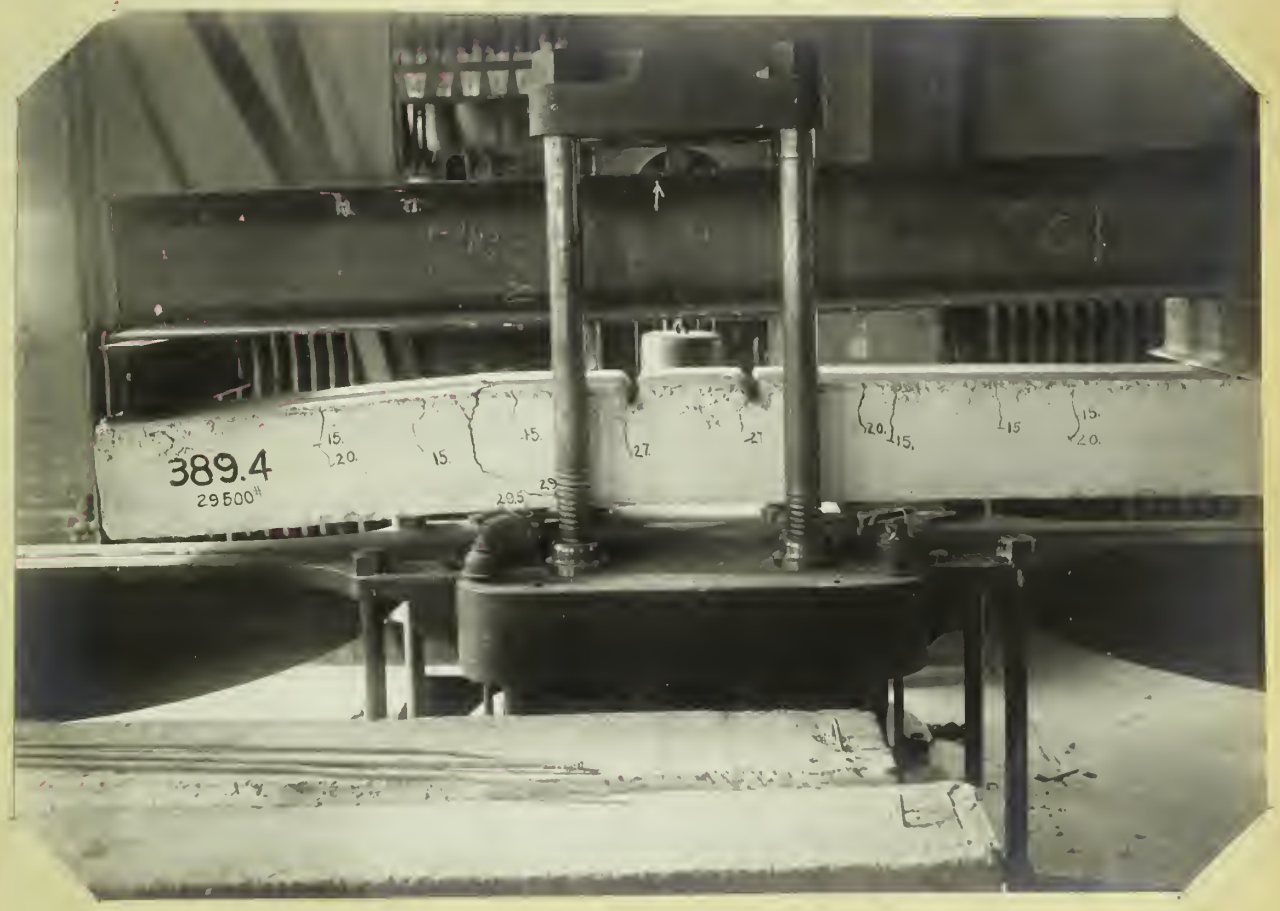
16. DISCUSSION.

Series I. While this thesis was under discussion the question was considered as to how the horizontal shear was affected by placing the bars in two layers and varying


389.1
23100#

389.2
31900#

389.3


389.4
29500#

the number of bars in each of the two layers but keeping the total number of bars constsnt. The horizontal shear was calculated for beams of 6 ft. span, 8 in. by 10 in. cross section loaded at the third points and containing 6 1/2 in. bars in two layers,, the number of bars in the top layer varying from 1 to 5 and vice versa in the bottom layer.

The horizontal shear was calculated from a formula

$$v'=uo/c \times (0.5+m'/m)$$

where

v'=horizontal shear.

u=unit bond stress.

o=circumference of bar.

c=clear space between bars in upper layer.

m'=no. of bars in the upper layer.

m=no. bars in the lower layer.

These results have been plotted and shown in Figure 21. From these curves it can be plainly seen that within the ordinary limits of design it makes practically no difference how the bars are distributed between the two layers as the horizontal shearing stress varies less than 14 per cent when the width of the beam at the plane of the steel is the same as the total width of the beam. When the width at the steel is 3/4 of the total width it makes no difference how the bars are distributed as the horizontal shear is the same in all cases. When the ratio a is more than 0.75 the smaller horizontal shear can be obtained by putting most of the bars in the bottom layer but when a is less than 0.75 the reverse is true and most of the bars should be in the upper layer. The latter condition however, rarely occurs in practical construction as the width

of the beam at the steel is almost always the same as at the top of the beam.

Because the theoretical variation in the horizontal shear due to the different placing of the bars in the two layers was so small it was not deemed expedient to make any tests along that line so the beams were all designed with the bars in one layer and at different intervals apart. It was found that by decreasing the width of the beam at the plane of the steel sufficiently high horizontal shearing stresses could be obtained.

Because of the small number of beams tested in this first series and the somewhat widely varying results for the beams of the same kind, it is rather difficult to draw many conclusions. Beams 381.1; 381.2; 385.1; 385.2; and 386.1;386.2 were the only beams that broke at aproximately the the same loads for similar pairs. The other four pairs broke at loads differing among themselves, the higher load being from 1.4 to 2.0 times the lower load. These different results were probably due to a variation in the quality of the concrete and the success in filling the space between the bars with the concrete. It seems to the writer that at least three beams of each variety should have been made and possibly more, but the time was so limited when the thesis was begun that much that could be wished for had to be left to some other time.

In none of the beams was the steel stressed beyond the elastic limit, the average calculated stress being about 24000 lbs. per sq. in.

No measurements as to the slipping of the bars were

taken but none was observed before failure and the deflection diagrams would indicate that none had occurred. In some cases at failure the bars pulled out as much as 1/4 in. but there is no indication that this occurred before the maximum load.

Series II. Bending up the reinforcing bars near the load points as can be seen from Figure 34 greatly increases the bond stress. Probably the bond strength is much larger for that section of the bars that is imbedded in the upper part of the beam or the shorter bars would fail in bond before the others. In not a single case of this series was there a failure due to bond though in one case, Beam No. 388.4 the unit bond stress at the elastic limit of the steel was about 580 lbs. per sq. in. The fact that the upper part of the beam has large compressive stresses probably accounts for the very large bond strength obtained.

It is rather remarkable that there was so little variation between the stresses in the bars of different lengths, in fact there does not seem to be much more variation in the stresses of the bars when they are bent up than when they are continuous. This is plainly seen in Figures 30 to 33 inclusive. Where a large difference between two bars runs through all of the loads as between bars 4 and 5 in Beam No. 389.3 it is probably due to an error in taking the zero reading on the extensometer.

In Figures 22 to 29 inclusive it can be seen that the curves are of the same general shape although there are very many variations among them even for the same beam. The curves for those beams that are alike and were loaded in the same manner are very similar as Figures 28 and 29. The difference

in shape and slope of the curves for Beams 389.1 and 389.2 Figures 26 and 27, respectively is due to the fact that they were loaded differently,; similarly Beams 388.3 and 388.4, Figures 24 and 25.

All of the beams developed aproximately the same resisting moment except Beam 389.3 which showed a strength about 25 per cent greater than the average.

From this series of tests it would appear that there is no danger of bond failure even if the bars are bent up directly under the load points although it is undoubtedly safer to bend them up at some distance out from the points of loading. It is probably safe to allow 1½ to 2 times the usual bond for bars that are bent up so as to pass through the compression side of the concrete.

17. SUMMARY.

Series I. Summing up the results of this thesis it would seem

1. The effect of position of the reinforcing bars is not great on the amount of the horizontal shearing stress when there is more than one layer of bars though a lower stress may be had by putting most of the bars in the lower layer or layers.

Within the range of these tests;

2. The strength of the beams increases directly as the average spacing of the bars.

3. As between two sizes of bars, the preference should be given to the larger bar where bond is not a consideration.

Series II. Within the range of these tests

1. The stresses among the several bars of the beam are fairly evenly distributed even when the bars are bent up at different intervals.

2. The bond strength of the bent up bars is greatly increased, an allowable bond stress of $1\frac{1}{2}$ to 2 times that ordinarily used for straight bars being safe.

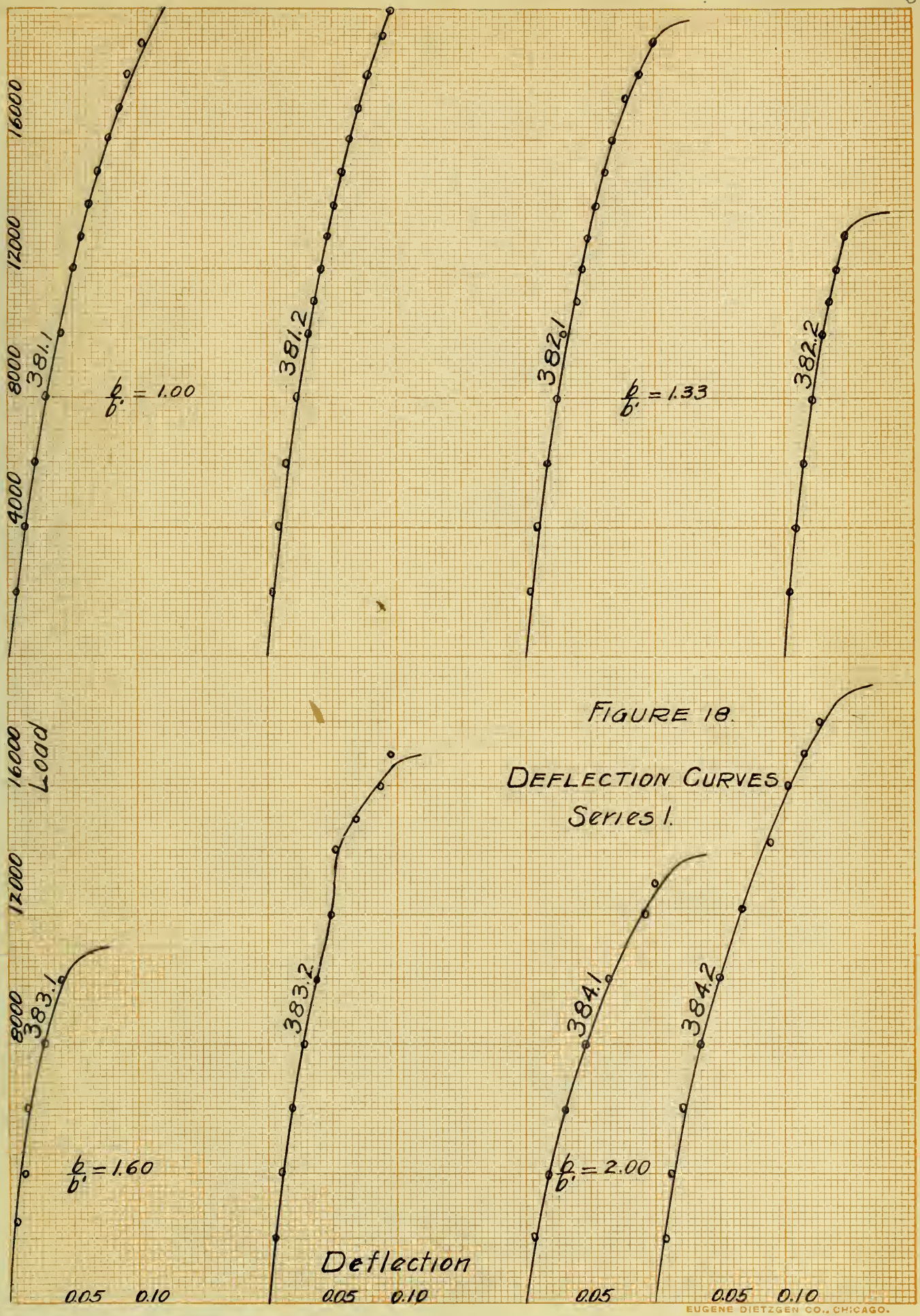


FIGURE 18.

DEFLECTION CURVES.

Series I.

EUGENE DIETZGEN CO., CHICAGO.

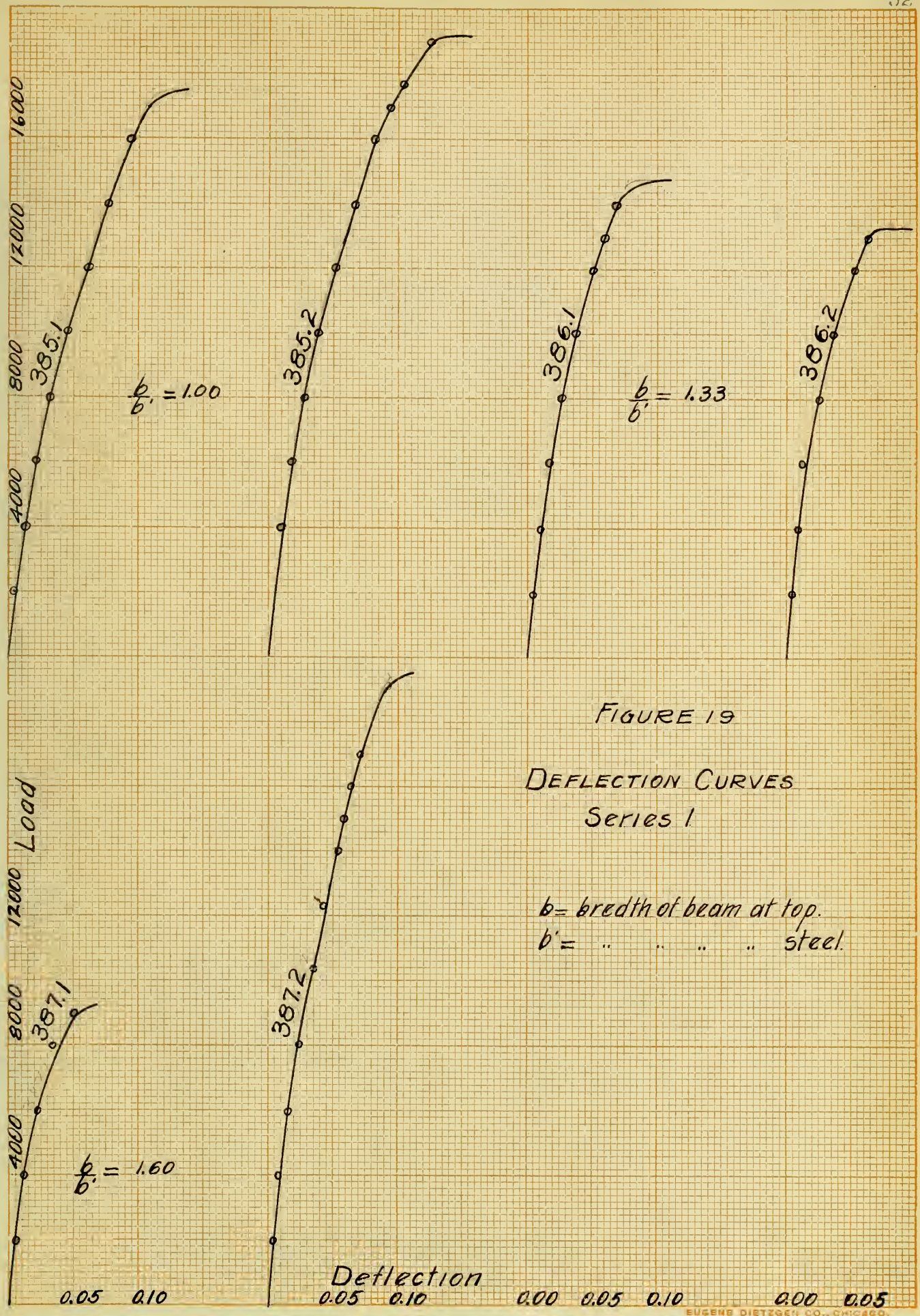


FIGURE 19

DEFLECTION CURVES
Series I

b = bredth of beam at top.
b' = " " " " steel.

EUGENE DIETZGEN CO., CHICAGO.

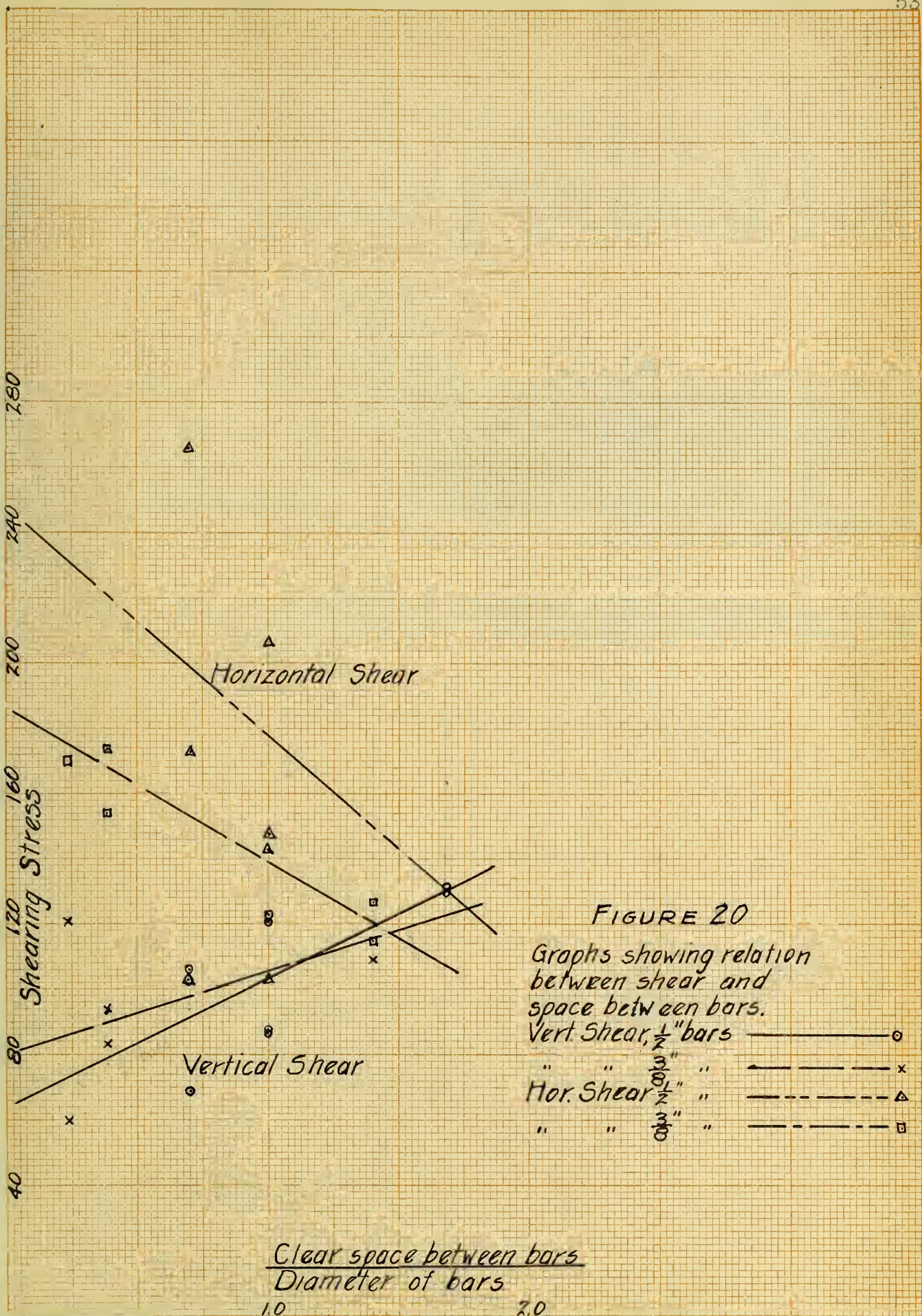


FIGURE 20

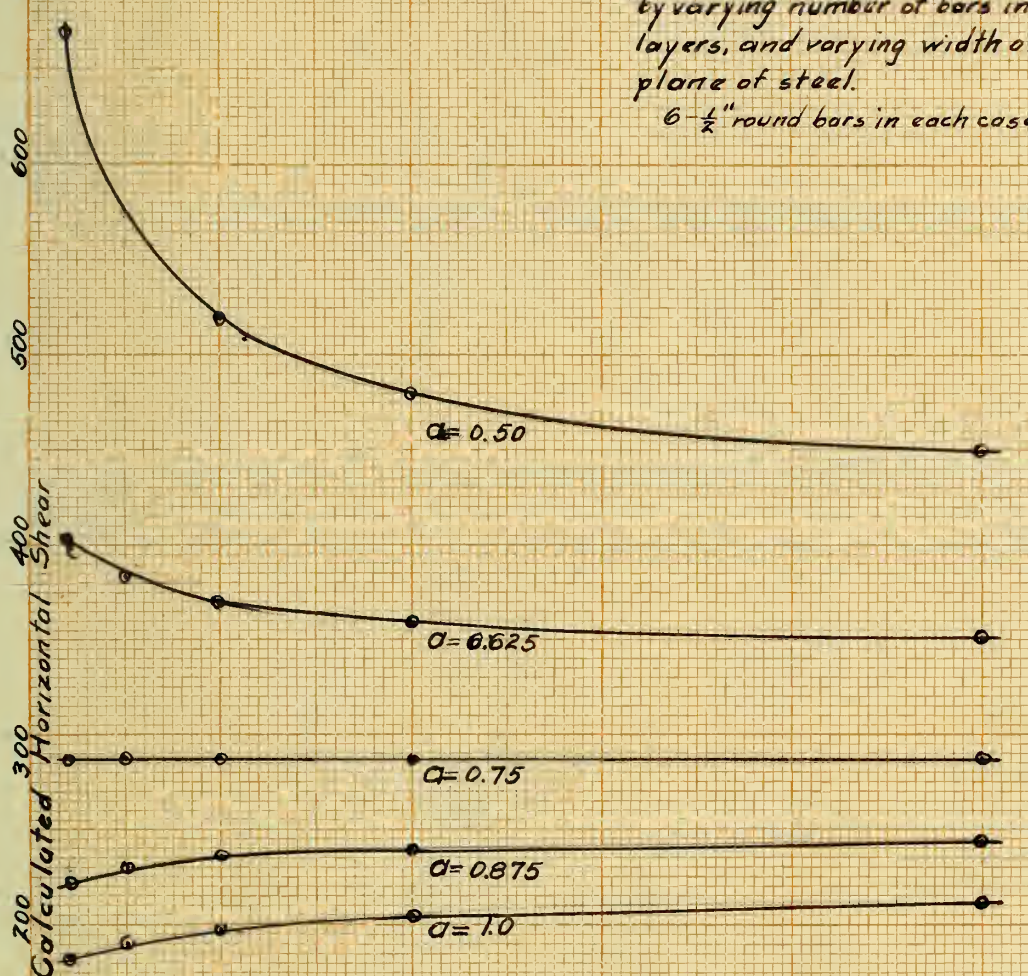
Graphs showing relation between shear and space between bars.

EUGENE DIETZGEN CO., CHICAGO.

FIGURE 21

Graphs showing effect on Horizontal Shear by varying number of bars in each of two layers, and varying width of beam at plane of steel.

6 - ½" round bars in each case.

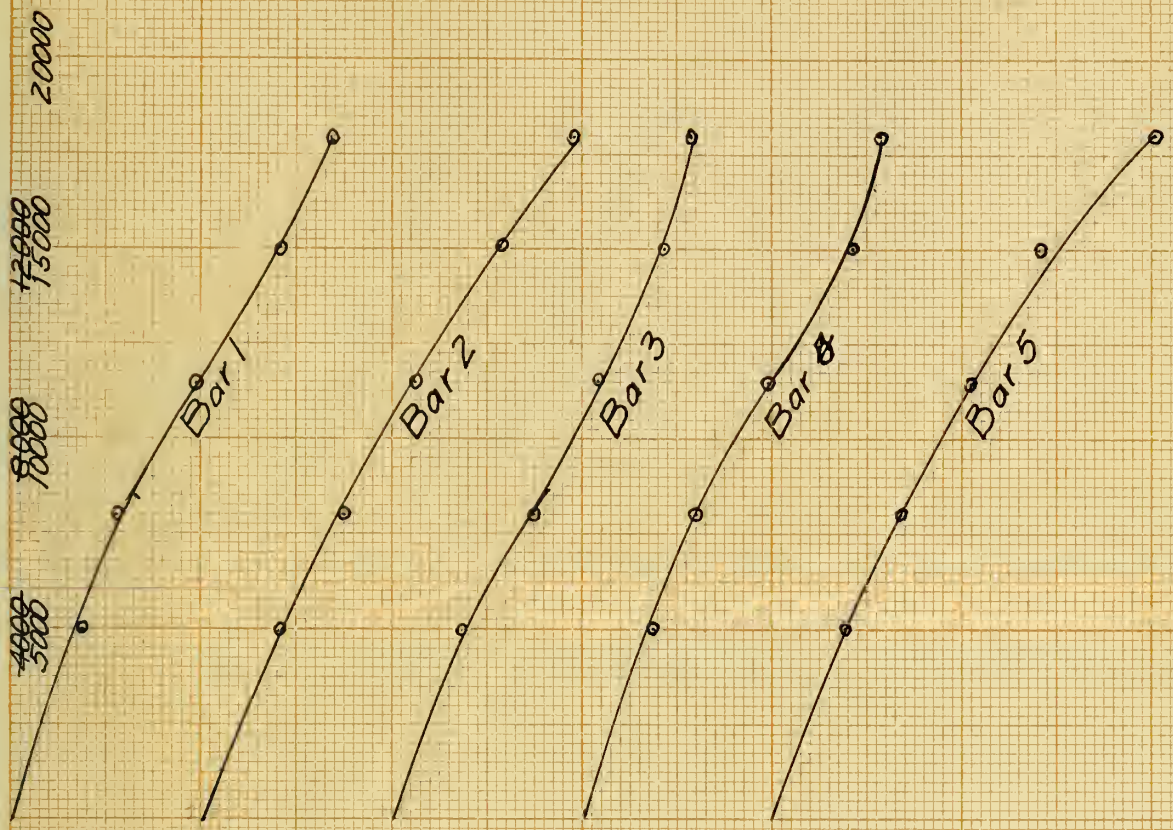


$a = \frac{\text{width of beam at steel}}{\text{width of beam}}$

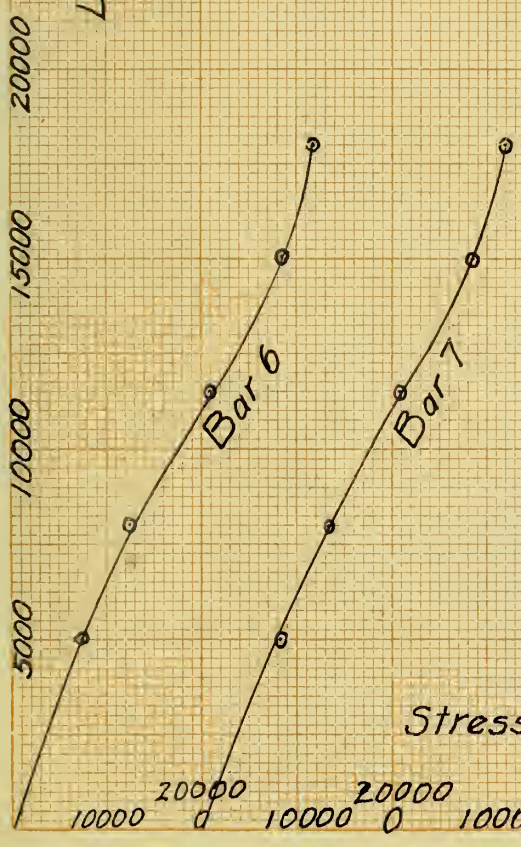
$K = \frac{\text{no. bars in upper layer}}{\text{no. bars in lower layer}}$

1.0 2.0 3.0 Values of K 4.0 5.0

EUGENE DIETZGEN CO., CHICAGO.



Load



Stress in steel

FIGURE 22
Beam 381.1
Graphs showing relation between load and stress in steel.

EUGENE DIETZGEN CO., CHICAGO.

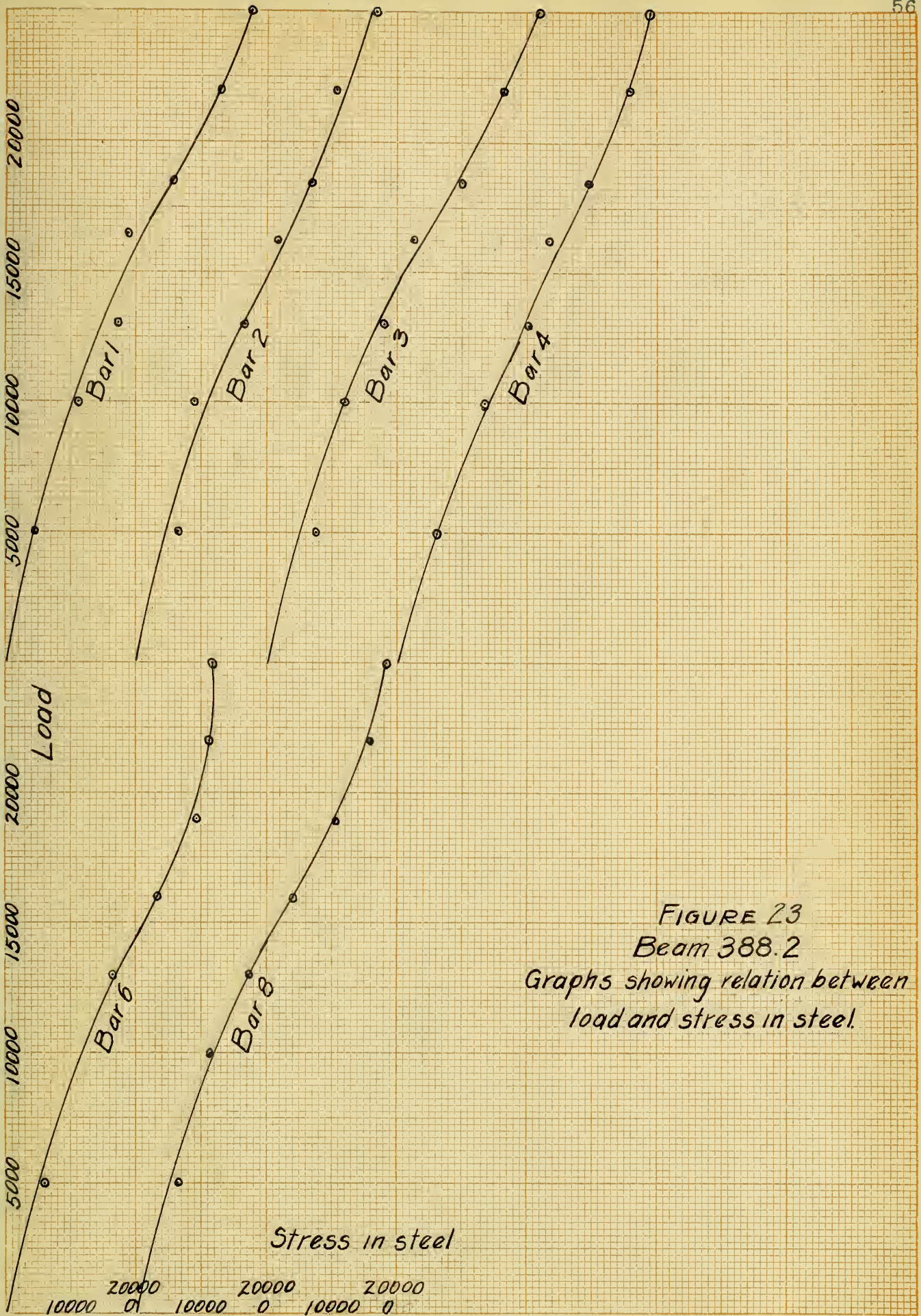


FIGURE 23
Beam 388.2
Graphs showing relation between load and stress in steel.

EUGENE DIETZGEN CO., CHICAGO.

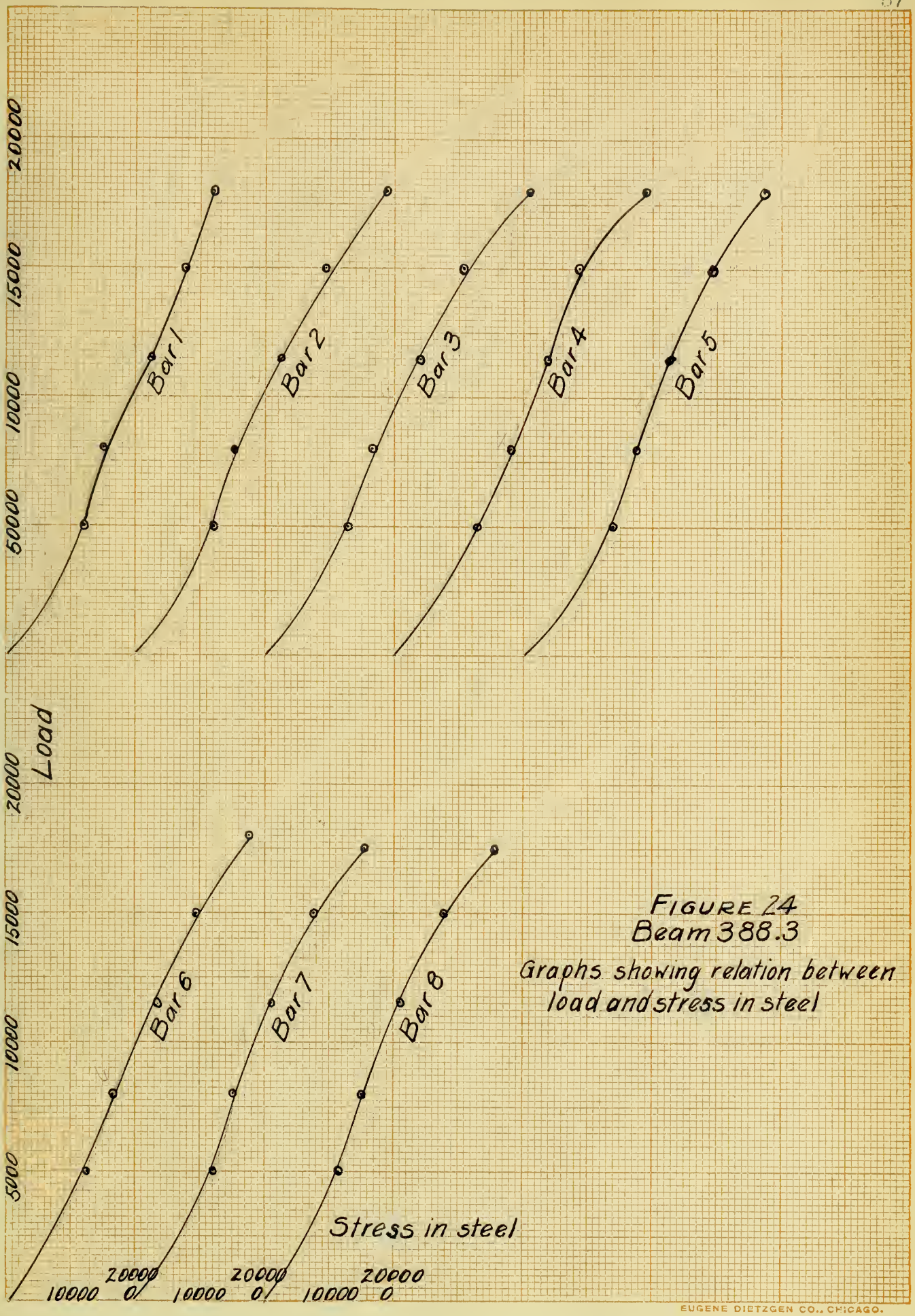


FIGURE 24
Beam 388.3
Graphs showing relation between load and stress in steel

EUGENE DIETZGEN CO., CHICAGO.

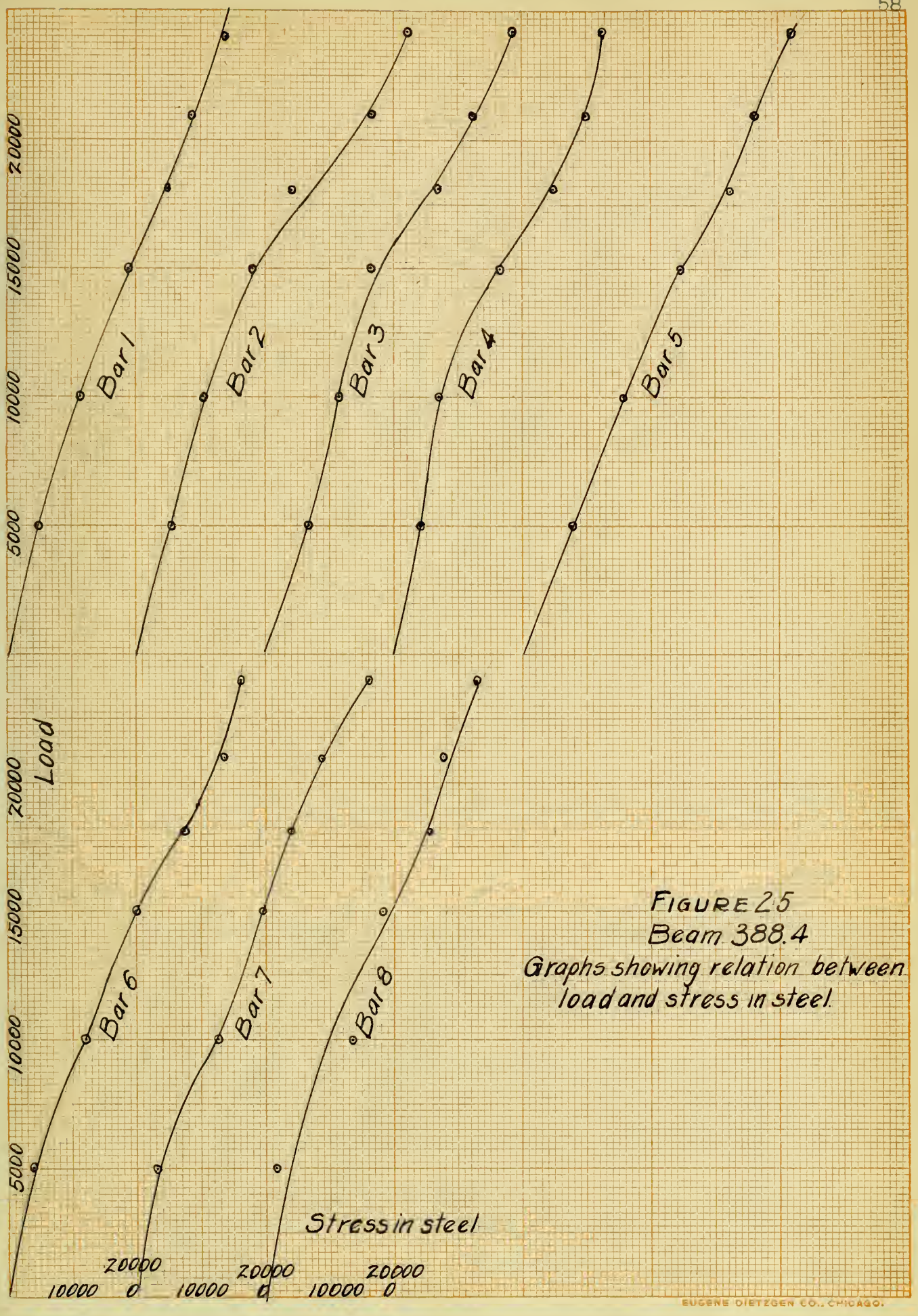


FIGURE 25
Beam 388.4
Graphs showing relation between load and stress in steel.

EUGENE DIETZGEN CO., CHICAGO.

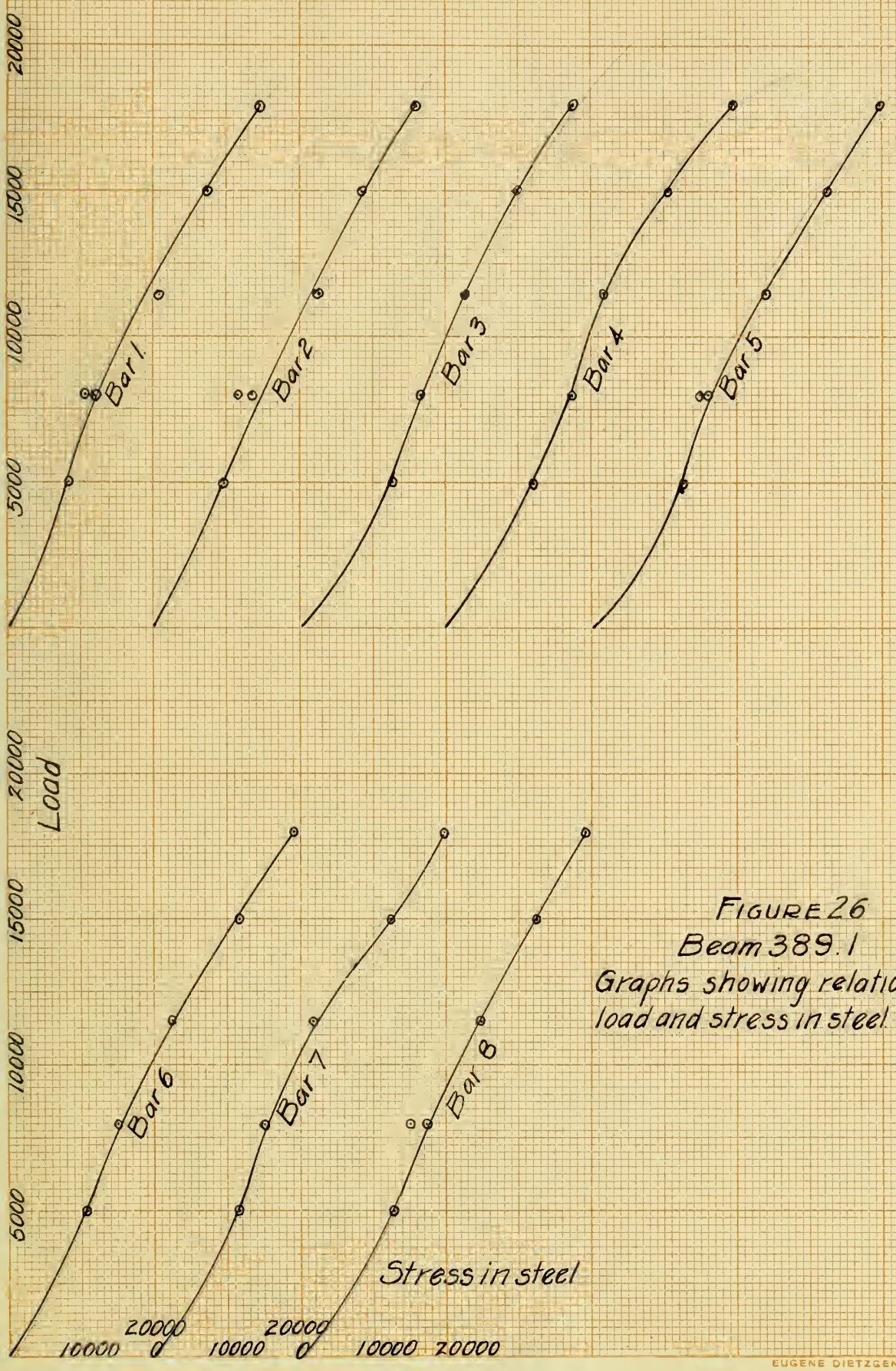


FIGURE 26
Beam 389.1
Graphs showing relation beteen load and stress in steel

EUGENE DIETZGEN CO., CHICAGO.

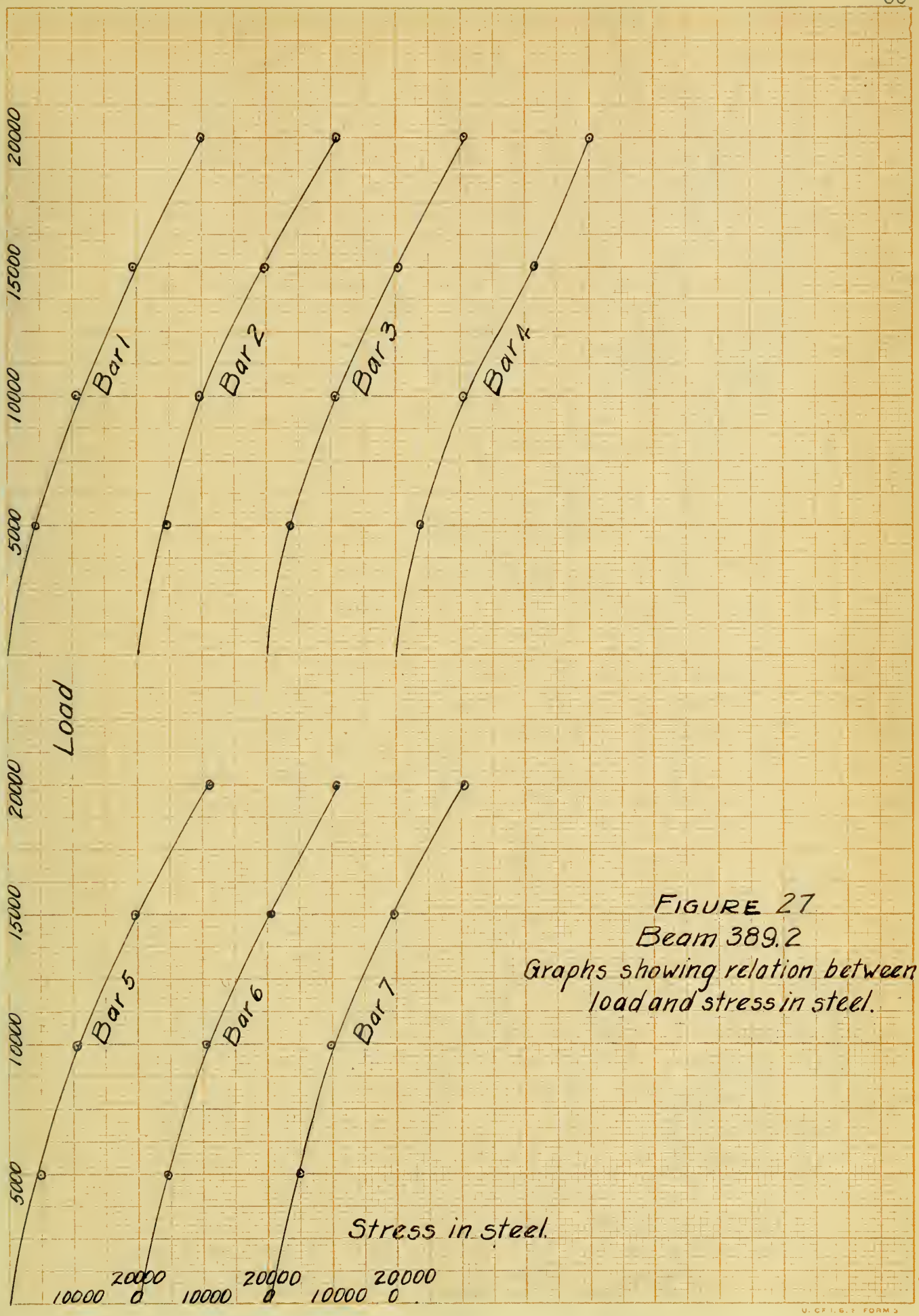


FIGURE 27
Beam 389.2
Graphs showing relation between load and stress in steel.

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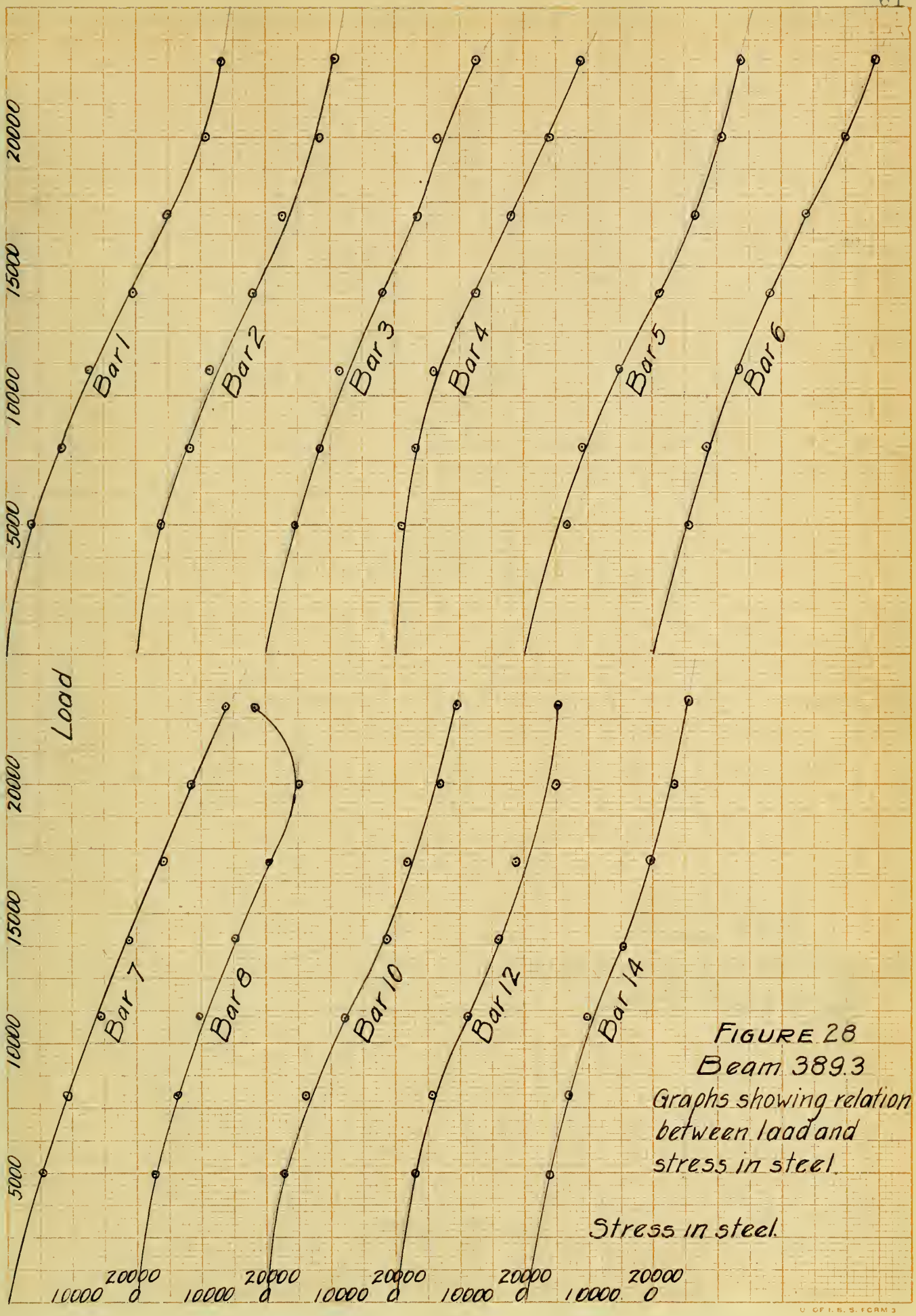

FIGURE 28
Beam 389.3
Graphs showing relation between load and stress in steel.

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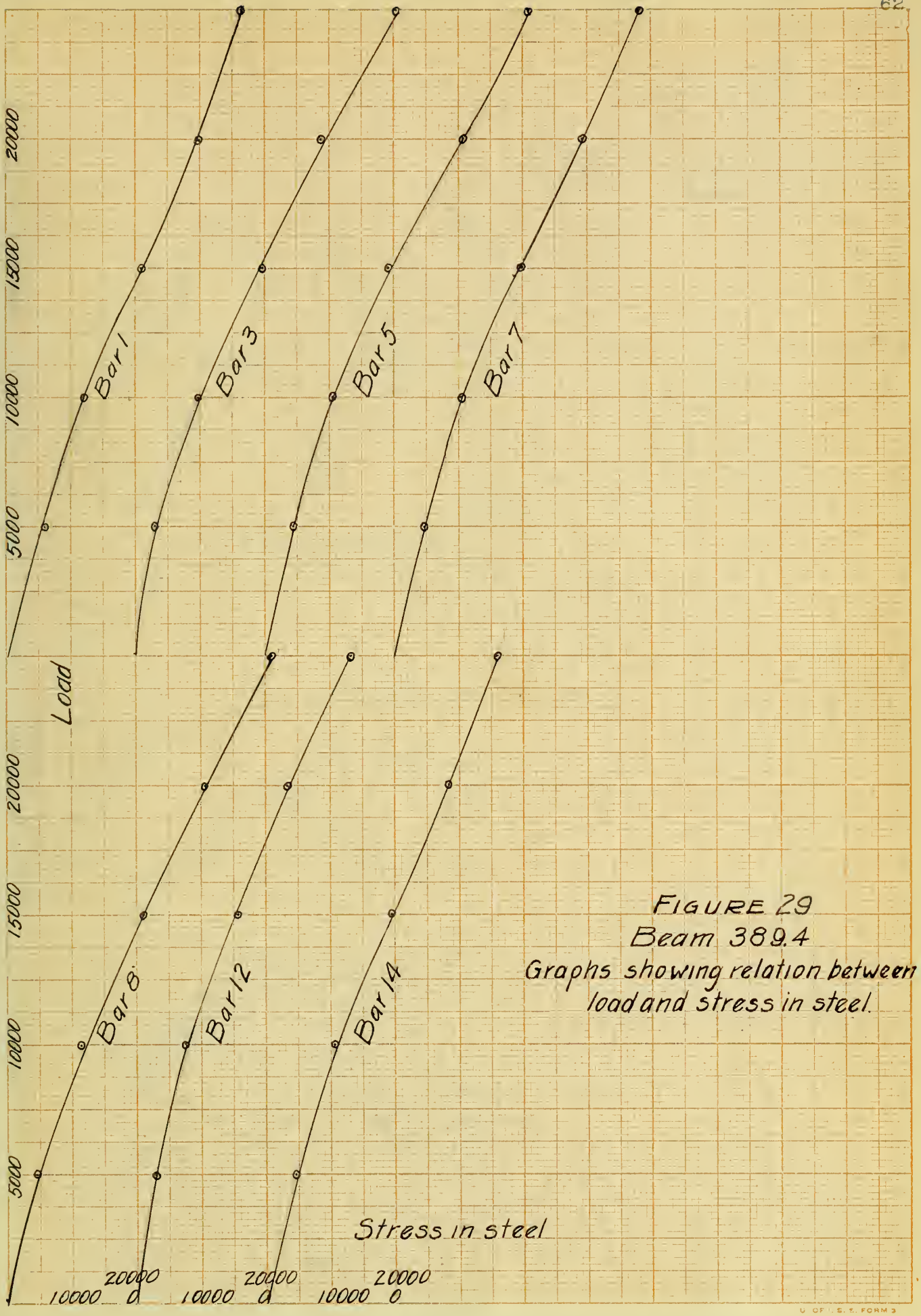

FIGURE 29
Beam 389.4
Graphs showing relation between load and stress in steel.

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FIGURE 30
Beam 388.2
Beam 388.1
Graphs showing stresses in the bars.

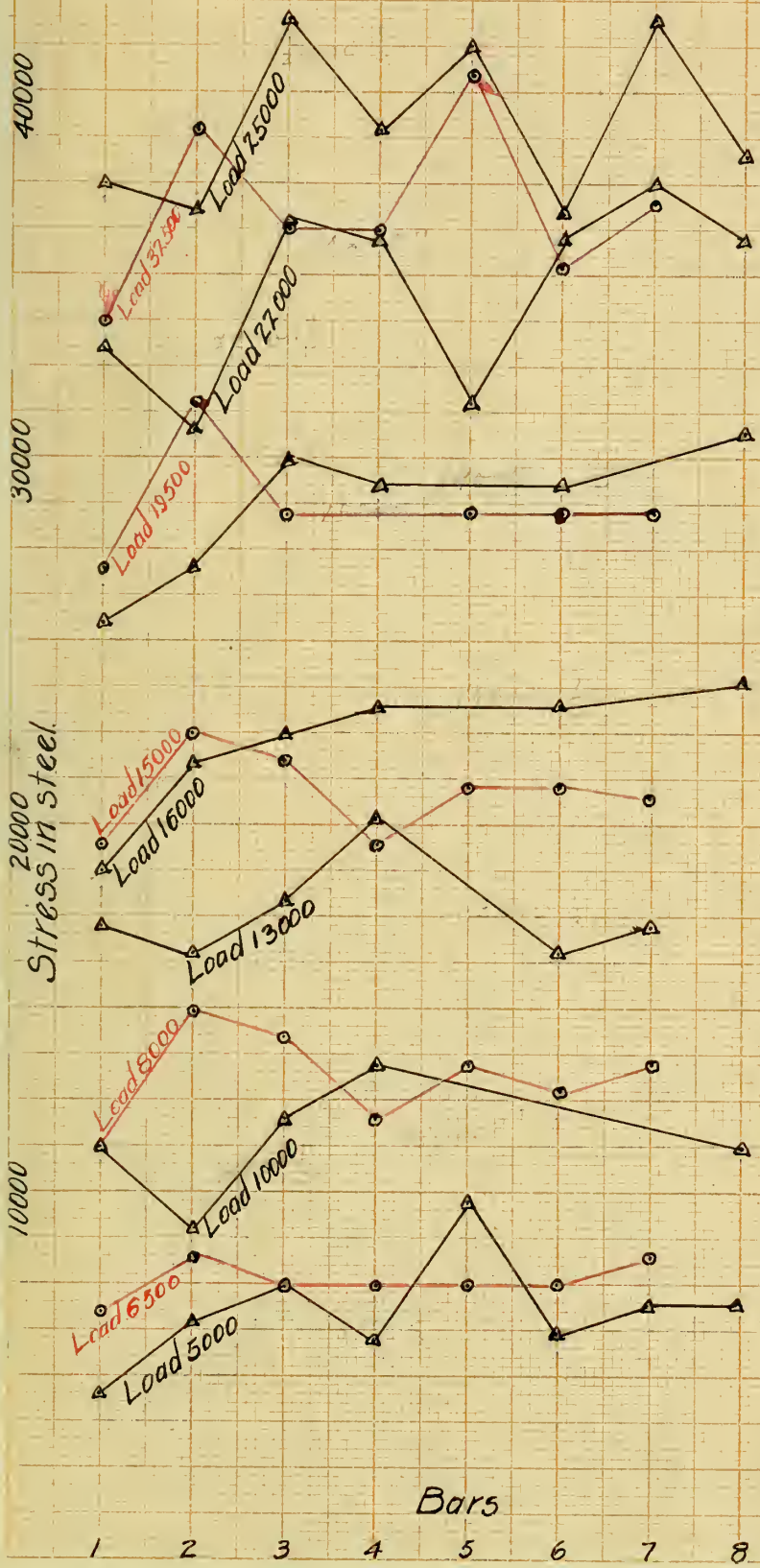

FIGURE 31
Beam 388.4
Beam 388.3
Graphs showing stresses in the bars.

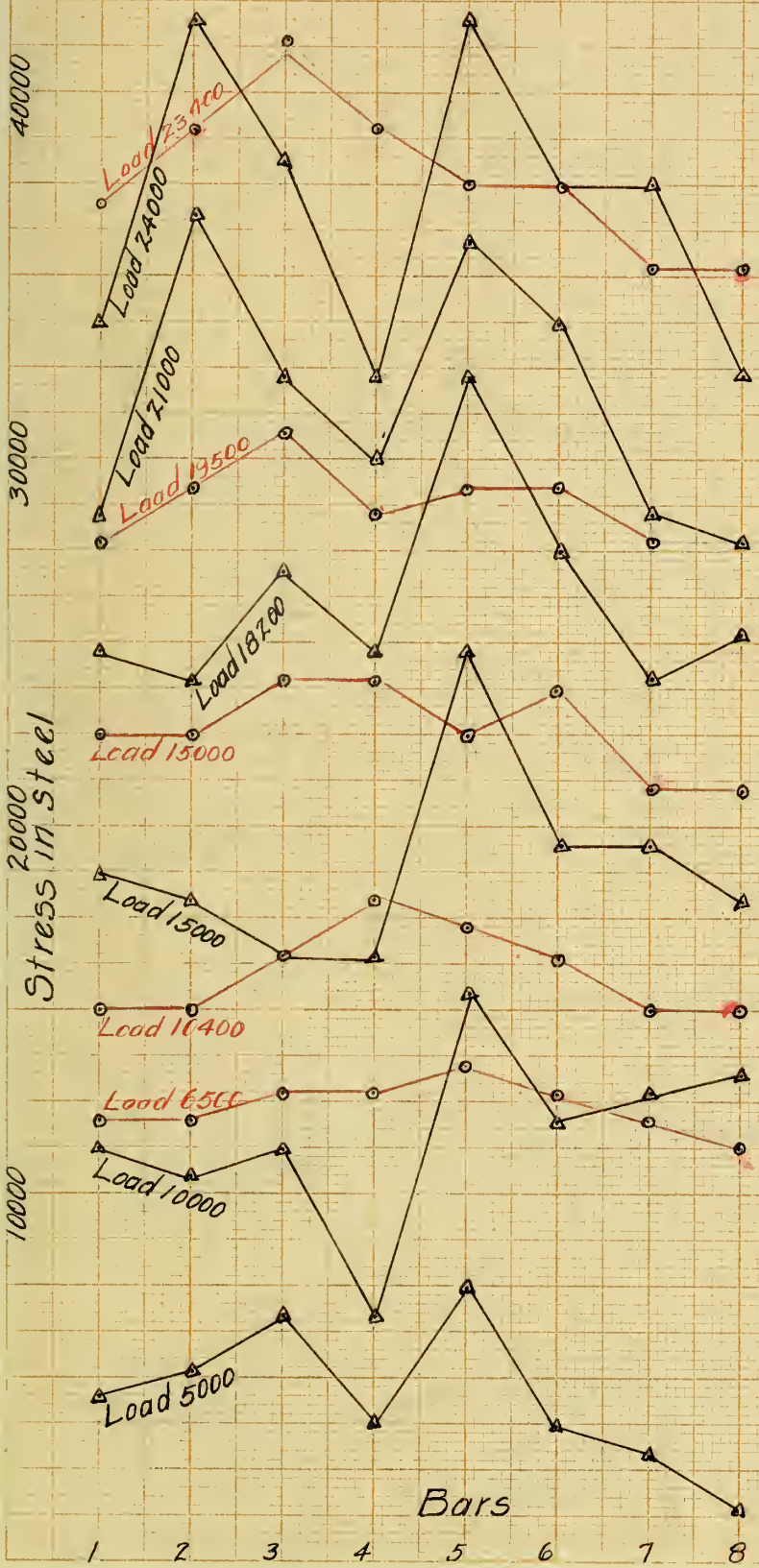

FIGURE 32
Beam 389. 2
Beam 389.1
Graphs showing stresses in the bars.

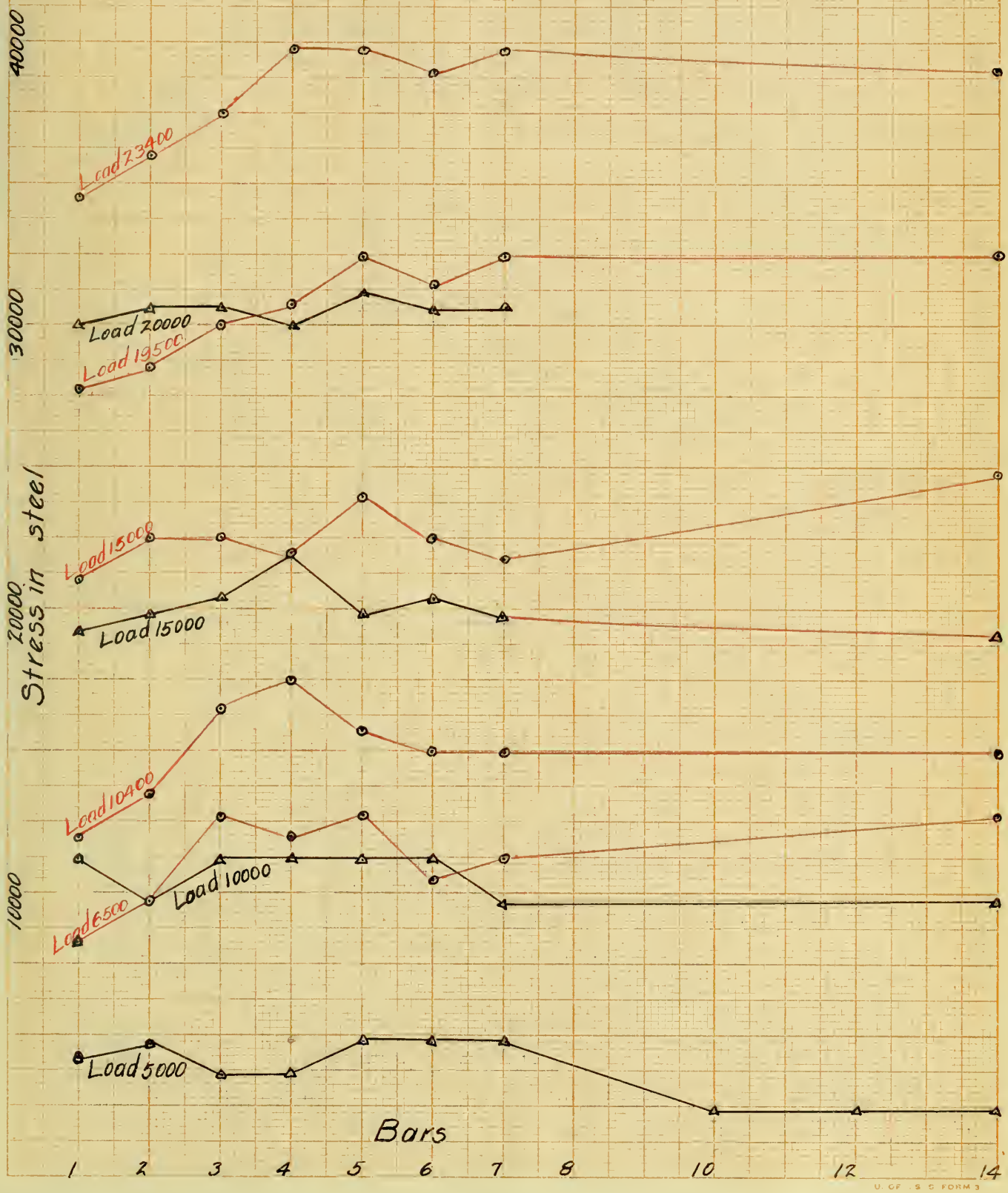

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FIGURE 33
Beam 389.4
Beam 389.3
Graphs showing stresses in the bars.

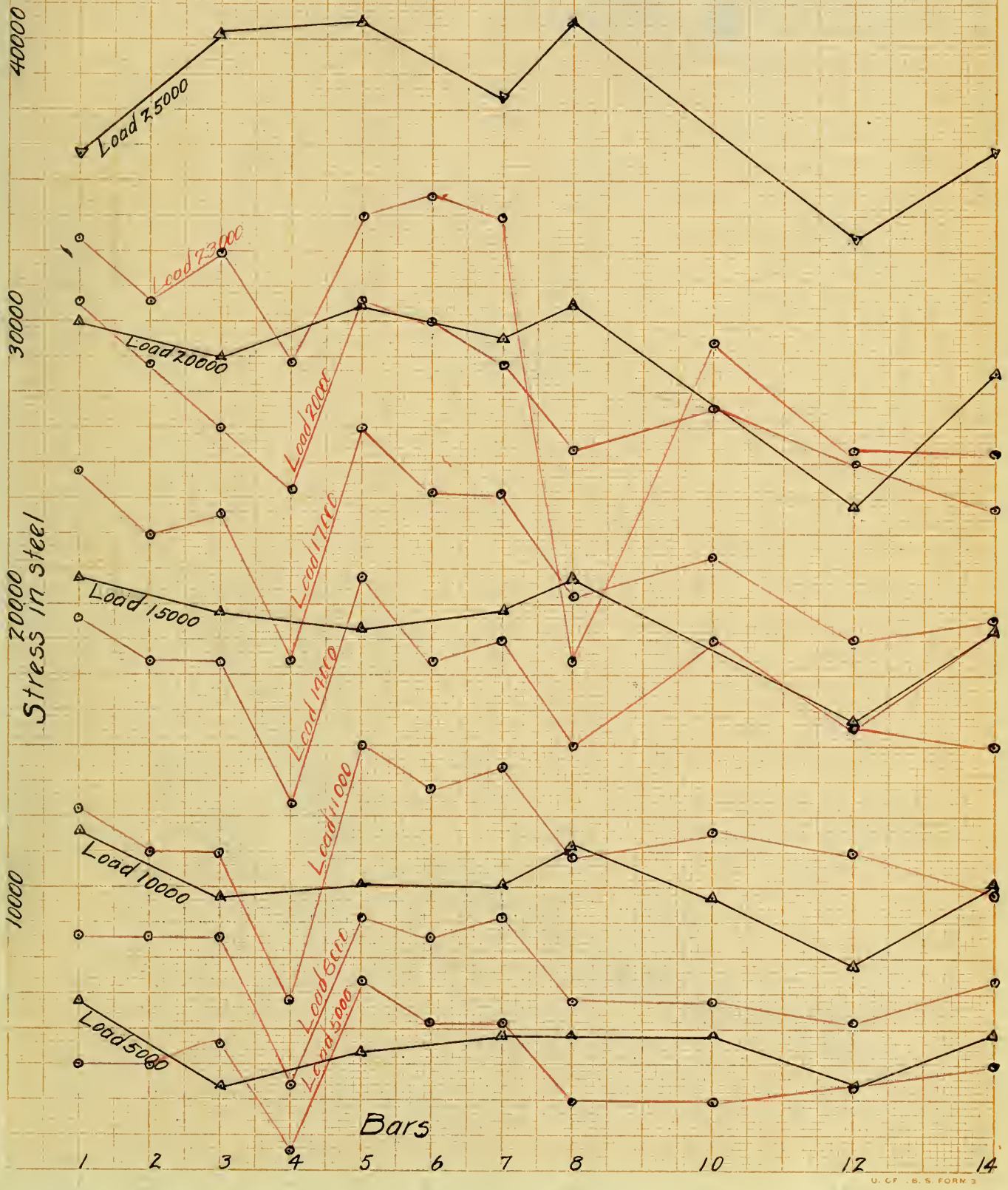

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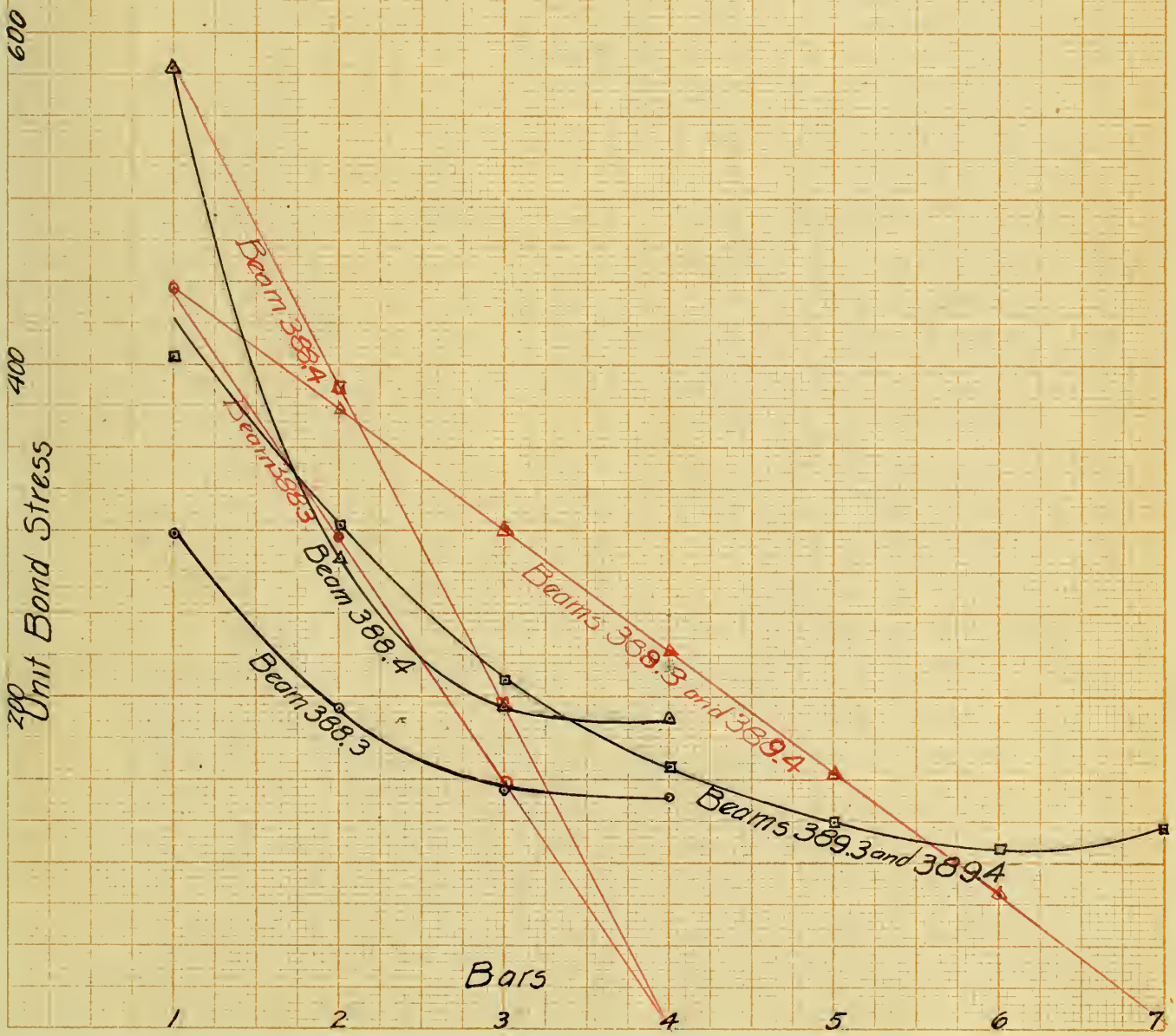
FIGURE 34
Graphs showing relation between unit bond and bars
Bar N°1 outside bar in each case.
Black graphs show average bond
Red graphs show maximum bond



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